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James Reuben Fincher

INVESTIGATION OF BASIC REQUIREMENTS FOR A
DIRECT MOMENT CONNECTION
USING HIGH-STRENGTH STRUCTURAL BOLTS AND WIDE FLANGE BEAMS

A THESIS

Presented to

the Faculty of the Graduate Division

by

James Reuben Fincher

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

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DIRECT MOMENT CONNECTION

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Approved:

Date Approved by Chairman: May 27, 1957

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF ILLUSTRATIONS

SUMMARY

The recent interest and study of the design of steel structures by plastic or limit design theory has focused attention on the action and design of moment connections. Not only must the connections have adequate strength to develop the plastic moment of members in a structure, but adequate rotation capacity must also be present to allow redistribution of moment. When to these requirements is added the necessity for economy of fabrication and erection, the combination of shop welding and field bolting using high-strength bolts shows particular promise. This combination has, in fact, been employed, but to the writer's knowledge only in connections designed so that the high-strength bolts develop the moment through shearing action, employing the friction developed by the high contact pressures between faying surfaces at working loads, and the actual bolt shear strength at ultimate load. Such a design, although superior to a riveted connection in many respects, does not take full advantage of the properties of high-strength bolts.

A study was thus undertaken to develop a moment connection for wide flange beams that would stress the high-strength bolts in tension, develop the full plastic moment of the beam, have adequate rotation capacity, be practical to fabricate and erect, and be capable of design by rational means. It was immediately apparent that the time and funds available would allow only a pilot program to be conducted.

A connection, called a direct moment connection, was devised, in which steel plates were welded to the end of two sections of a wide

flange beam. Matching holes were previously drilled in the connection plates and the splice was accomplished by placing in these holes high-strength structural bolts torqued to high initial tension. Five such beams with connections of varying strength were prepared and tested by loading the connection in pure moment. The test program consisted of three series of tests; the first using two 8 WF 20 beams in which the connection plate thickness was varied, the second using two 8 WF 17 beams in which the bolt size was varied, and a final test of an 18 WF 50 beam with a balanced design. Design of the connections tested was made using theoretical equations developed during this study.

Conclusions drawn from the data indicate:

(1) The direct moment connection as devised can be designed to develop the full plastic moment of wide flange beams, and will have adequate rotation capacity.

(2) Theoretical expressions developed are essentially correct.

(3) The fabrication and erection offer no special problems and, in fact, have many advantages over presently used connections.

(4) Action of the direct moment connections tested in this study warrants a complete investigation.

NOMENCLATURE

a	=	lever arm between resultant of contact pressure and resultant of bolt forces.
b	=	gross width either of beam flange or connection plate
b_f	=	specifically the width of the beam flange
b_p	=	specifically the width of the connection plate, may be either gross or net width as required
C	=	total contact force between connection plates of a direct moment connection
c	=	unit contact pressure between connection plates of a direct moment connection
c_i	=	initial unit contact pressure
c_m	=	unit contact pressure due to moment
c_t	=	maximum unit contact pressure on the tension side
c_c	=	maximum unit contact pressure on the compression side
d	=	depth of beam and/or length of connection plate
e	=	vertical distance from beam neutral axis to resultant of bolt forces
F_f	=	force in beam flange
h	=	portion of web acting with beam flange or connection plate to form "T" section
I_p	=	moment of inertia of faying surface of connection plate
M	=	moment in a beam
M_b	=	moment developed by bolt group
M_p	=	plastic moment of beam
M_{pf}	=	plastic moment of "T" section composed of beam flange acting with portion of web

M_{pp} = plastic moment of "T" section composed of connection plate acting with portion of web

M_u = ultimate moment

m = vertical distance from compression flange of beam to bolt nearest the tension flange

n = number of bolts in a single vertical row of a direct moment connection

P = axial force

p = pitch of bolts measured along vertical row

q = vertical distance from tension flange of beam to bolt nearest tension flange

R = number of rows, containing n bolts each, in a direct moment connection

T_1, T_2, T_3..... force in a particularly high strength bolt where T_1 is the bolt nearest the tension flange, T_2 the next above--the higher the subscript, the farther the bolt from the tension flange.

T_i = initial force in a high strength bolt

ΣT_i = summation of initial bolt forces for all bolts in a direct moment connection

ΔT_i = change in initial force in a high strength bolt

T_n represents the bolt in a vertical row farthest from the tension flange

T_u = ultimate force in a high strength bolt

t_f = thickness of beam flange

t_p = thickness of connection plate

t_w = thickness of beam web

V = total shear load carried by direct moment connection

V_b = shear load carried by direct moment connection by bolts in bearing

V_f = shear load carried by direct moment connection through friction

y = vertical distance measured from beam neutral axis

$y_1, y_2, y_3 \ldots\ldots$ vertical distance of a particular bolt measured from the tension flange. Subscript corresponds with bolt location as above for bolt forces

y_n = distance to the bolt in a vertical row farthest from the tension flange

$\bar{y}$ = vertical distance from the tension flange to the resultant of bolt forces

ϵ = unit strain of a high strength bolt

ϵ_u = ultimate unit strain of a high strength bolt

σ_y = yield stress

σ_{yf} = yield stress of beam flange

σ_{yp} = yield stress of connection plate

σ_{yw} = yield stress of beam web

μ = coefficient of friction

θ = angle of rotation of plastic hinge associated with failure of connection plate

ϕ = unit rotation

ϕ_y = unit rotation of beam when outermost fiber reaches yield

CHAPTER I

INTRODUCTION

The design of connections has always been one of the major sources of difficulty in structural engineering. Evidences of the dilemma are the large number of tests conducted through the years, the inconclusive and often conflicting data obtained from these tests, and sections of the present design codes which are based as much on opinion and judgment as on scientific fact. Trends to the use of continuous and rigid-frame structures have compounded the problem of connection design.

However, the formation of the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation in 1947 has led to important advances in connection design. Notable among these is the development of structural high-strength bolts. These bolts are only now coming into everyday use; and, at present, field applications are restricted to a one-for-one replacement of rivets. Further, the bolts are used in connections which are a replica of riveted connections. The high-strength bolt is severely penalized when used merely as a replacement for a rivet in a conventional riveted connection; particularly so when designed to be a moment connection, as full advantage cannot be taken of the bolt's tensile strength. This fact alone demands a serious effort to devise a moment connection more suited to high-strength bolts.

When one considers the requirements of a moment connection that will be satisfactory for use in structures designed by the theory of plastic analysis, the desirability of a new approach is strengthened. A moment connection, when used in structures designed by plastic analysis, must have, in addition to strength, sufficient rotation capacity (1).[1] Any connection devised must also be practical from the standpoint of shop fabrication and field erection. The following points have been selected as desirable criteria for a moment connection using high-strength bolts:

(1) Adequate strength

(2) Rotation capacity

(3) Shop welded

(4) Field bolted

(5) Bolts in tension

(6) Simple to fabricate

(7) Easy to erect

(8) Fits into the overall structural frame with ease

(9) A pleasing appearance

The moment connection shown in Fig. 1 is the result of a study of the above requirements. It is entirely shop welded and field bolted. The bolts act in tension to carry both moment and shear.[2] This type connection will hereafter be referred to as a direct moment connection or simply a direct connection. It is felt that the last four requirements are fulfilled in a manner superior to that of any presently used connection.

Tests were run to evaluate the first two requirements. Since no

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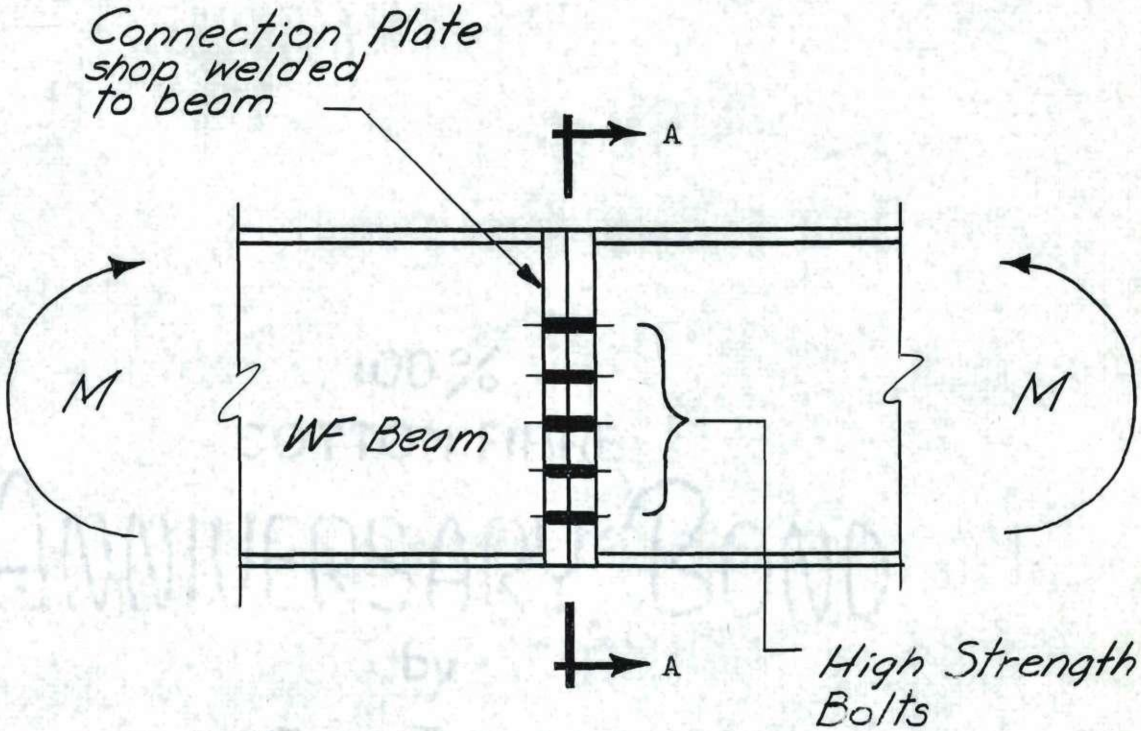
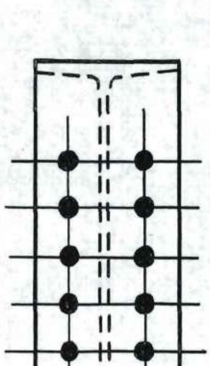
[1]Numbers in parentheses indicate references listed in the bibliography.

[2]The exact action of the connection is explained in detail in Chapter II.



Elevation



Section A-A

Fig. 1 Direct Moment Connection

previous data have been published on a connection of this type, the tests were designed as a pilot program. The time and funds available would not allow a program extensive enough to answer all the pertinent questions relative to details. Rather than confuse the issue with attempts to study a large number of variables, the program was limited in scope and designed primarily to answer two questions: 1) Will the general response of the connection be adequate? 2) Can the action be predicted from simple theoretical considerations?

CHAPTER II

THEORETICAL CONSIDERATIONS

Definition and Properties of High-Strength Bolts

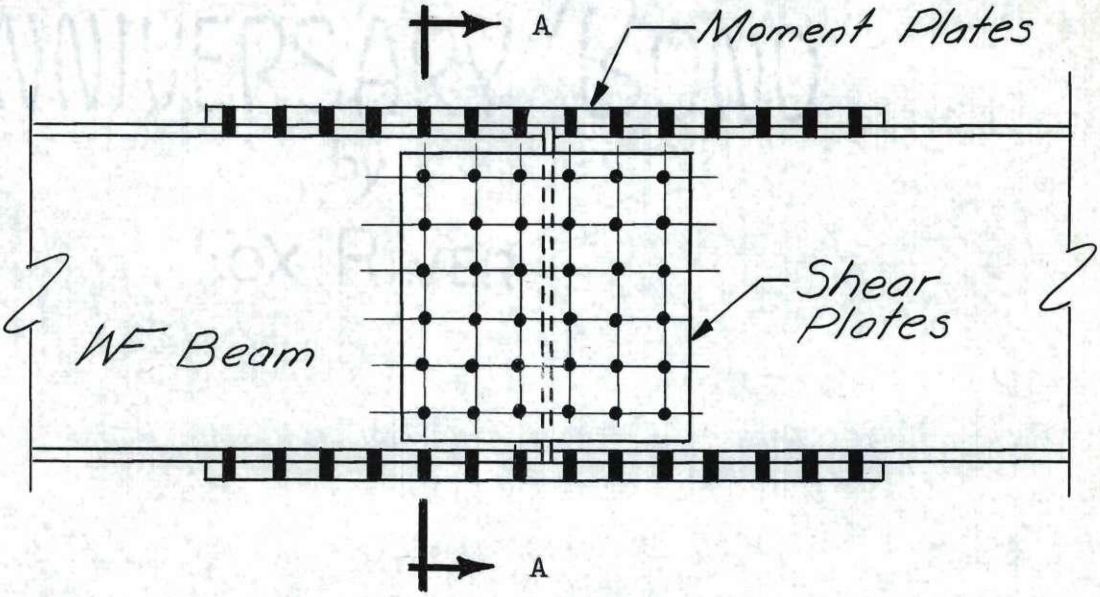
High-strength bolts as used in structural steel connections conform to "Tentative Specifications for Quenched and Tempered Steel Bolts and Studs with Suitable Nuts and Plain Hardened Washers," American Society of Testing Materials designation A325-55T. These specifications establish minimum requirements for strength, hardness, and ductility. In addition, assembly of high-strength bolts is controlled by "Specifications for Assembly of Structural Joints Using High Strength Steel Bolts," approved February, 1954 by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation and endorsed by the American Institute of Steel Construction.

The minimum ultimate stress requirements vary with the bolt size, decreasing for increasing diameter. For 7/8-inch diameter bolts, this minimum ultimate tensile stress is 115,000 pounds per square inch. In practice the bolts are tightened to 90 per cent of the elastic proof load. This proof load also varies with the bolt diameter, and for the 7/8-inch diameter bolt is based on a stress of 78,000 pounds per square inch. The bolt must withstand the specified proof load with a maximum permanent set of 1.2 per cent of the thread pitch, or stated another way, 0.2 per cent of the length occupied by six full threads.

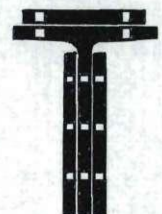
Due to the high stresses employed, high-strength bolts establish a relatively large clamping force when installed in a connection; for a

7/8-inch diameter bolt this clamping force is at least 32,400 pounds. This contact force can be used advantageously in several ways. If a connection is loaded normal to the bolt axis, the transfer of load between faying surfaces may be carried by friction at working loads, and the bolts installed in one-sixteenth inch oversize holes will not come into bearing. Should the connection be loaded past the designed working load, slip may occur; but only enough to allow the bolts to come into bearing. At this time the bolts will act in the same manner as is assumed in the design of riveted joints; however, the friction will still be present and the bolts carry in bearing only the overload. Both laboratory and field tests have shown that there is no tendency for the bolts to loosen during service, even under repetitive loading (2). Thus even when loaded in shear, high-strength bolts offer a considerable increase in carrying capacity over the same size rivet because the known contact force makes possible the elimination of bearing consideration at working loads. The bolts also provide a larger reserve of ultimate capacity due to the combination of high friction and shear, even after slip.

If, however, the working load is to be based on no slip, as is common practice, the full effectiveness of the bolt cannot be realized as the friction can only be a fraction of the contact force. Consideration should be given to loading the bolts along their axis to increase the carrying capacity of the joint. While this practice may not be feasible for connections that are stressed primarily in shear, it can readily be accomplished for moment carrying connections. The present typical splice for a wide flange beam develops both shear and moment capacity through the fasteners' shear strength as shown in Fig. 2 (3);



Elevation



Section A-A

Fig. 2 Typical Splice for Wide Flange Beam Under Current Practice which not only requires many fasteners, but loose plates during erection, and a reduction in area of the section due to the flange holes. Because of the reduction in area the designer usually locates splices in regions of low moments-- feasible in structures where the dead load to live load ratio is high, but troublesome if the inflection point shifts. Since in continuous and rigid frame structures, moments normally control the design; the major emphasis on a splice should be the development of the moment capacity. As will be apparent from the explanation of the action of a type connection as shown in Fig. 1, the full shear capacity of the section may be developed at the same time.

Theoretical Expressions for Connections Using High-Strength Bolts

For axial loads.--From the two plates, shown in Fig. 3a, which are held together by four high-strength bolts, principles of basic mechanics will readily show that the following statements are true if the load P acts at the centroid of the bolt group:

The plates will not separate until the applied external force exceeds the initial contact force, or separation is imminent if the external force equals the initial tension.

$$P - nRT_i + C = 0 \; *$$

or $$C = 0 \quad \text{when} \quad P = nRT_i = \Sigma T_i \qquad (1)$$

The applied external force will not alter the initial bolt force nor its internal stress, assuming a rigid plate, as long as the applied

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*Signs for all equations are referenced to the faying surface of the connection plate. Forces acting in a direction away from this surface are considered positive.

force is less, or at the most, equal to the summation of all bolt tensions.

$$\text{If} \qquad P \leq \Sigma T_i \quad \text{then} \quad \Delta T_i = 0 \tag{2}$$

Should the applied external force exceed the initial bolt force, the difference will be carried as an increase in bolt tension. In effect, the total force that may be carried is simply the ultimate capacity of the bolts. The initial bolt tension merely controls the plate separation as far as axial loads are concerned.

The shear capacity may best be described using separate expressions. If no axial external load is present, then the external shear capacity will be simply the total bolt tension times the appropriate coefficient of friction.

$$\text{For} \qquad P = 0 \quad \text{then} \quad V_f = \mu\, RnT_i = \mu\, \Sigma T_i \tag{3}$$

If an external axial load is present, then the contact pressure between plates is altered and therefore the shear capacity may either increase or decrease, depending on whether the axial load is compression or tension.

$$\text{For} \qquad 0 \leq P \leq \Sigma T_i$$

$$V_f = \mu\,(nRT_i \pm P)$$

$$V_f = \mu\,(\Sigma T_i \pm P) \tag{4}$$

Once the external shear force exceeds the value of either of the above cases, the bolts come into bearing and the capacity will be a summation of the appropriate case above plus the shear or bearing capacity of the bolts as is presently used in riveted designs.

$$V = V_f + V_b \tag{5}$$

These equations may better be understood from the freebodies in Fig. 3b, 3c, 3d and 3e.

These equations would, of course, still hold if the plates were welded to the ends of a member; a wide flange beam for example, and the external force applied through the member, assuming that the connection plates have sufficient strength to transfer the load.

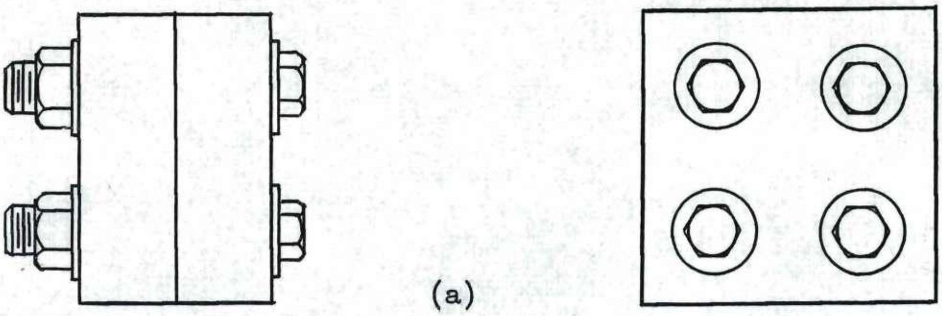
<u>For moment</u>.-- Suppose that instead of an axial load a moment exists on the member. Equations may be developed for this case, also using only the relations of statics, by superimposing the initial contact pressure due to the fasteners and the contact pressure due to the applied moment. Pressure due to fastener tensions will be the summation of the initial bolt tensions divided by the plate area:

$$c_i = \frac{nRT_i}{bd} = \frac{\sum T_i}{bd} \tag{6}$$

Pressure due to the applied moment can be determined at any point in the same manner as soil pressure for a footing under eccentric loading, and the extreme contact pressures due to an applied moment can be found from the following equations:

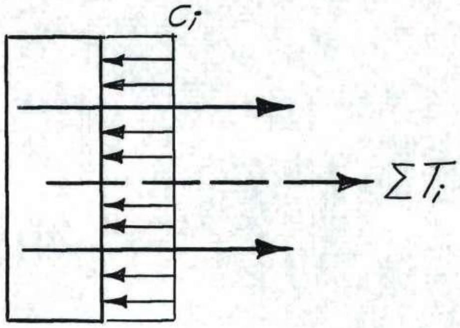
$$c_m = \pm \frac{My}{I_p}$$

$$c_m = \pm \frac{Md}{2\left(\frac{bd^3}{12}\right)} = \pm \frac{6M}{bd^2} \tag{7}$$



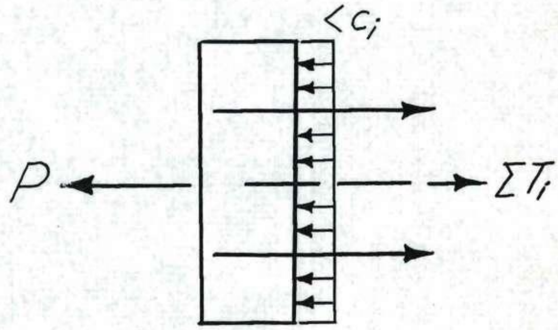
(a)

Plates Clamped with High Strength Bolts



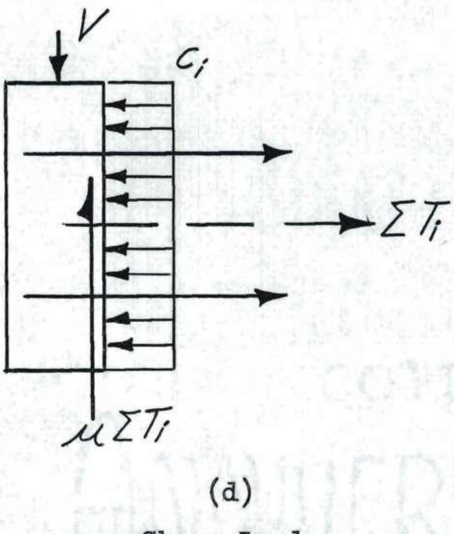
(b)

Zero External Load



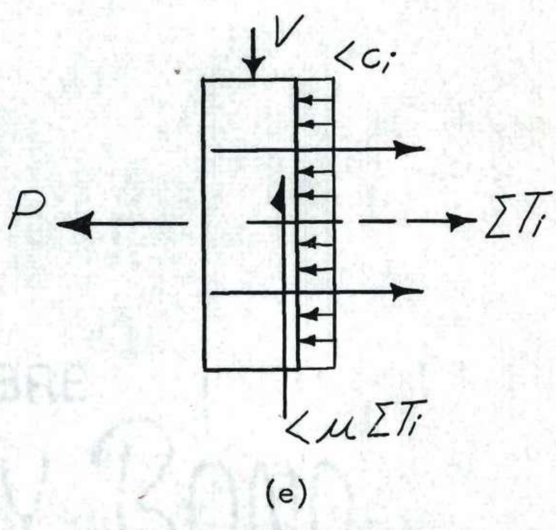
(c)

Axial Load



(d)

Shear Load



(e)

Axial + Shear Load

Fig. 3 Freebodies of Plates Clamped with High-Strength Bolts Under Various Loads

An expression for the resulting maximum or minimum contact pressure of a symmetrical connection, having initial bolt tension acted on by an external moment, may now be stated:

$$c = c_i + c_m$$

$$c = \frac{\Sigma T_i}{bd} \pm \frac{6M}{bd^2}$$

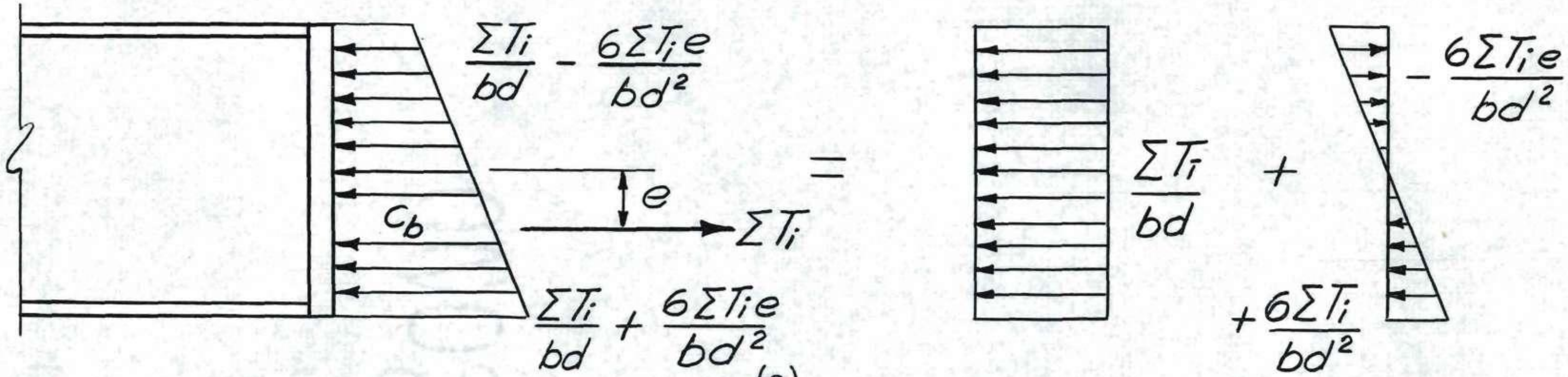
$$c = \frac{1}{bd}\left(\Sigma T_i \pm \frac{6M}{d}\right) \text{ where } \Sigma T_i \geq \frac{6M}{d} \qquad (8)$$

Equation (8) does not hold when $\frac{6M}{d}$ exceeds ΣT_i because, assuming a rigid connection plate, once the initial contact pressure reaches zero at any point between the plates, further loading will cause the bolt tensions to change.

From Equation (8) it can be seen that zero contact pressure will occur for a symmetrical connection when the applied external moment equals the summation of the initial bolt tensions times one-sixth of the connection plate depth. If zero contact pressure is taken as an upper limit for working loads, to keep down excessive deflections due to plate separation, then the next logical step would be to employ an unsymmetrical bolt arrangement. By shifting the bolts toward the tension side of the beam, the connection can be made even more efficient. From the free-bodies of Fig. 4 a relation for such a situation may be developed.

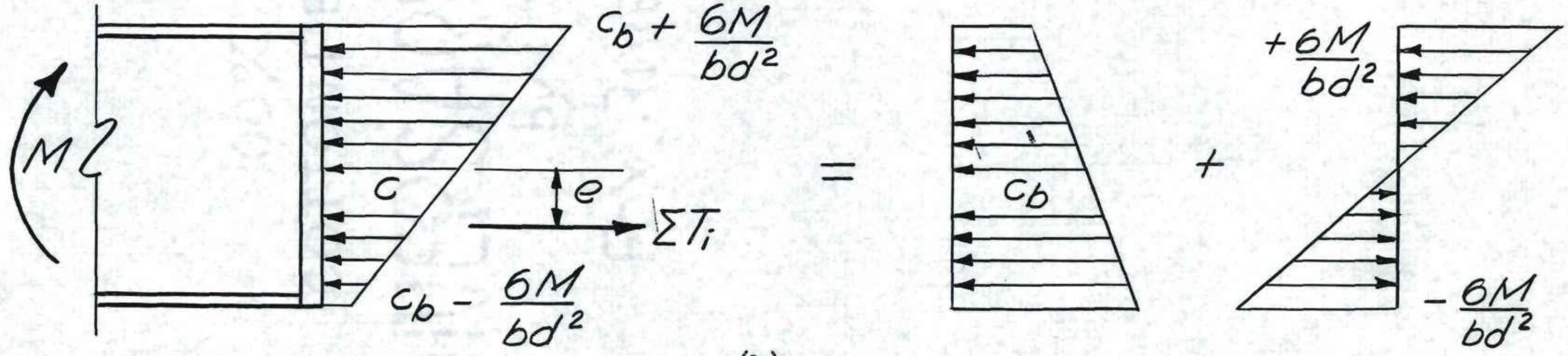
$$c = c_i + c_m$$

$$c_b = \frac{\Sigma T_i}{bd} \pm \frac{6 \Sigma T_i e}{bd^2}$$



(a)

Before Application of External Moment



(b)

After Application of External Moment

Fig. 4 Contact Pressures for Direct Connection with Eccentric Bolt Arrangement and ΣT_i within Kern

$$c_m = \frac{6M}{bd^2}$$

$$c = \frac{\Sigma T_i}{bd} \pm \frac{6 \Sigma T_i e}{bd^2} \pm \frac{6M}{bd^2}$$

$$c = \frac{1}{bd}\left[\Sigma T_i(1 \pm \frac{6e}{d}) \pm \frac{6M}{d}\right] \text{ for } \frac{d}{3} \leq \bar{y} \leq \frac{2d}{3} \qquad (9a)$$

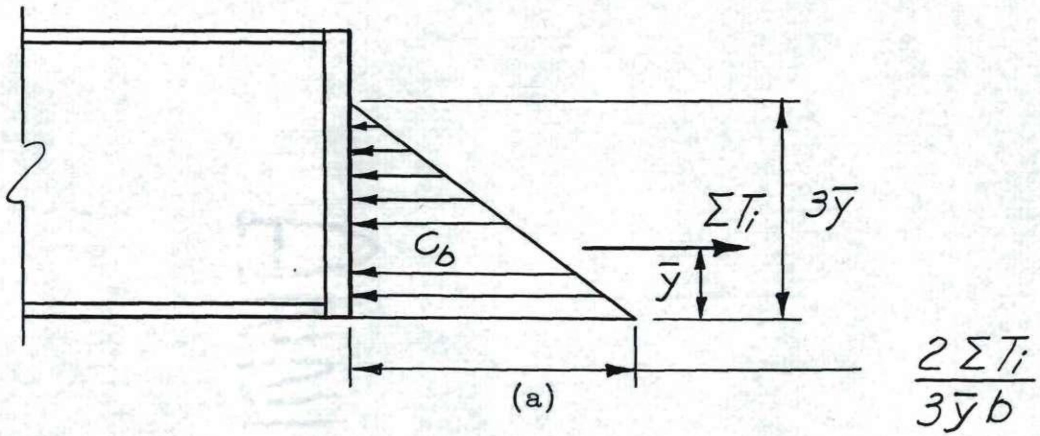
This expression applies only so long as ΣT_i falls within the middle third of d, or within the kern limit.

When ΣT_i lies outside the kern limit, the freebodies of Fig. 5 are typical; and from these, separate expressions for the contact pressure on the tension and compression side may be developed:

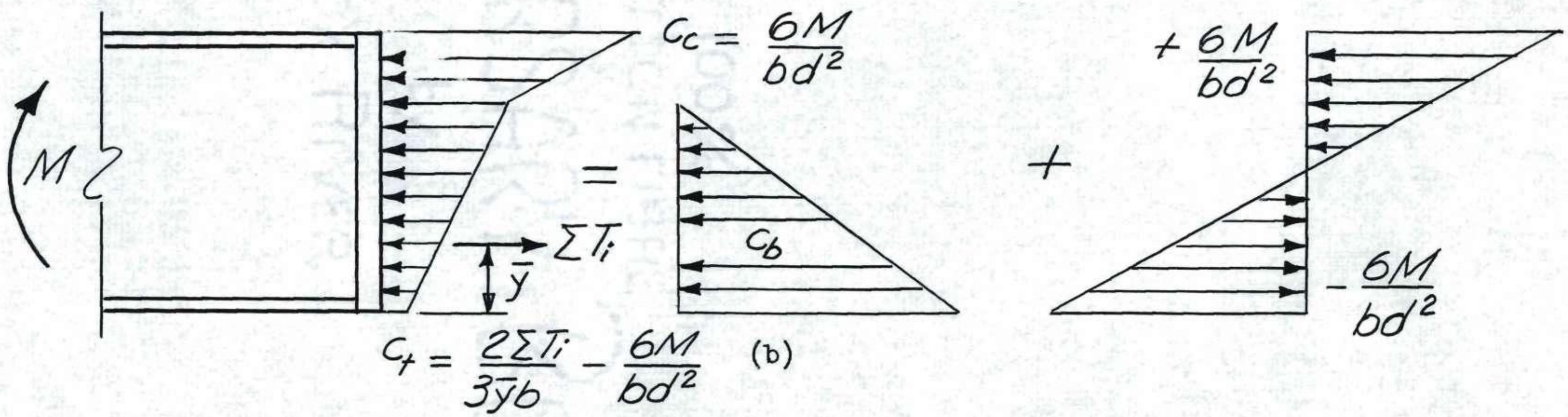
$$c_t = \frac{2}{3} \quad \frac{\Sigma T_i}{b\bar{y}} - \frac{6M}{bd^2} \quad \text{for } \frac{d}{3} > \bar{y} \qquad (9b)$$

$$c_c = \frac{6M}{bd^2} \quad \text{for } \frac{d}{3} > \bar{y} \qquad (9b')$$

An unsymmetrical bolt arrangement has the effect of decreasing the initial contact pressure on the side of the connection away from the direction of eccentricity and increasing the initial contact pressure on the eccentric side. Thus, an additional moment may be applied before loss of contact; the ratio of increase over a symmetrical connection is represented by $1 + \frac{6e}{d}$ for the same number of fasteners, when ΣT_i lies within the kern, as will be the case for most practical applications. This represents a valuable increase in carrying capacity by raising the allowable moment at working load if loss of contact is used as the limiting criterion. Of course, this eccentric arrangement of the bolts would



Before Application of External Moment



After Application of External Moment

Fig. 5 Contact Pressures for Direct Connection with Eccentric Bolt Arrangement and ΣT_i outside Kern not be desirable if the member were subjected to equal alternating moments of opposite sign. Fortunately, in most practical moment carrying members, the moment, even when varying from positive to negative or vice versa, has one predominant direction. Thus, the connection may be designed eccentrically for the largest moment and checked against loss of contact for the opposite but lesser moment.

It should be noted that theoretically the total initial contact pressure is not altered by application of an external moment less than the moment required to cause loss of contact. A larger moment will cause loss of contact progressively from the tension side, but the bolt tensions will increase at the same time. The total contact pressure will also increase by the same amount. Thus, theoretically the friction force available for shear resistance can never be reduced by the application of moment to the member.

Once sufficient moment has been applied to the member to cause loss of contact on the tension side of the connection, the stress in the bolts will increase, as stated above. The exact relation of this increased bolt stress will depend on the connection plate stiffness, which in turn is affected by the web stiffness of the beam. If the end connection plate is infinitely stiff, then the connection will rotate about the top, and bolt strains will increase in proportion to their distances from the top of the beam. Under such a condition the bolt farthest from the top will reach its ultimate load first and fail. The bolt immediately above would have a stress very close to ultimate at the time the lower bolt reached ultimate, and the sudden failure of the lower bolt would cause a chain reaction, passing the load upward, reducing the effective lever arm, and failing each successive bolt in turn. This will happen very suddenly and it can be stated that the ultimate strength of such a connection is reached when the bolt or bolts farthest from the compression side reach their ultimate capacity.

An expression for this case, assuming a rigid plate, may also be derived from the free body shown in Fig. 6. A necessary condition of a rigid plate would be rotation about the uppermost point on the compression side. At such a point the compressive stress due to the moment would be infinite, but the rigid plate is in itself impossible and is here used only as a step in the prediction of the action of a direct connection.

$$C = (T_1 + T_2 + \ldots\ldots T_n)R$$

$$M = C(\frac{d}{2} + e) = \left[(T_1 + T_2 + \ldots\ldots T_n) \quad (\frac{d}{2} + e)\right] \quad R$$

where $$e = \frac{d}{2} - \bar{y}$$

and $$\bar{y} = \frac{T_1y_1 + T_2y_2 + \ldots\ldots T_ny_n}{T_1 + T_2 + \ldots\ldots T_n}$$

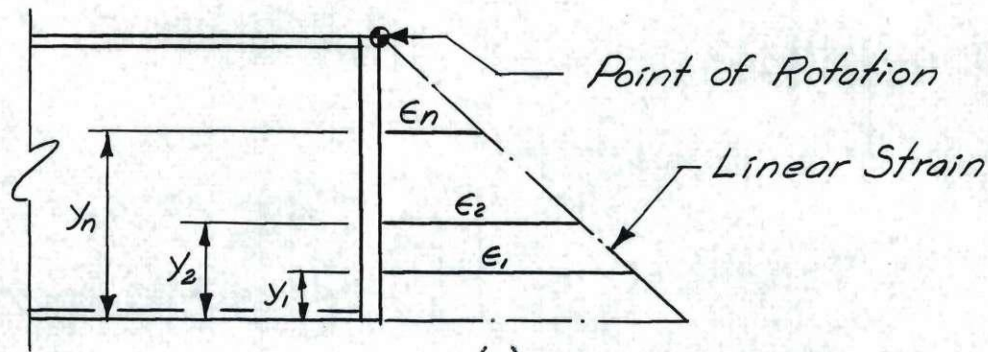
$$M = \left[(T_1 + T_2 + \ldots\ldots T_n) \left(d - \frac{T_1y_1 + T_2y_2 + \ldots\ldots T_ny_n}{T_1 + T_2 + \ldots\ldots T_n}\right)\right] \quad R$$

$$M = \left[d(T_1 + T_2 + \ldots\ldots T_n) - (T_1y_1 + T_2y_2 + \ldots\ldots T_ny_n)\right] \quad R$$

also

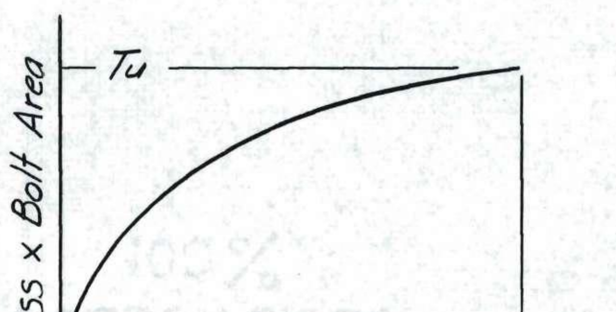
$$M_u = \left[d(T_u + T_2 + \ldots\ldots T_n) - (T_uy_1 + T_2y_2 + \ldots\ldots T_ny_n)\right] \quad R \qquad (10)$$

To evaluate the ultimate moment capacity of the connection from



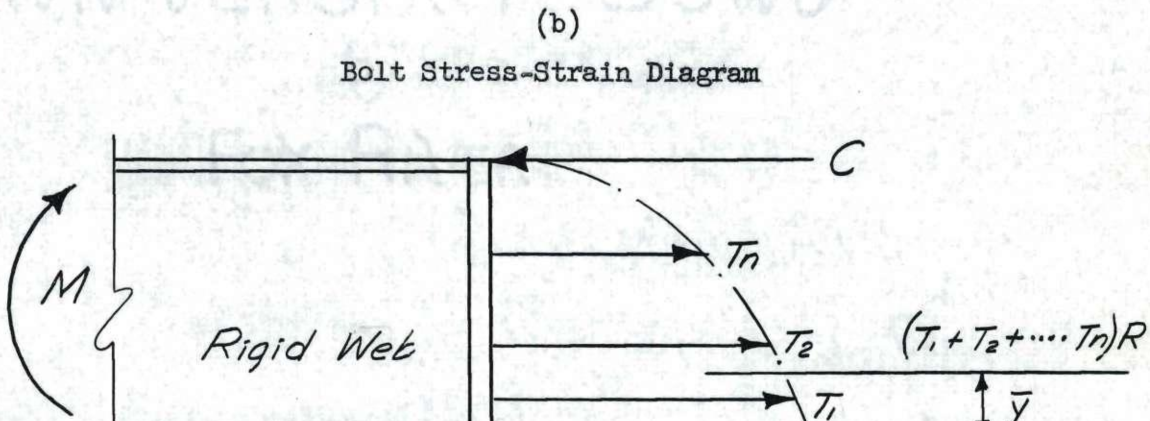
(a)

Strain Relation



(b)

Bolt Stress-Strain Diagram



(c)

Force Relation

Fig. 6 Action of Bolts in Direct Moment Connection with a Rigid Plate

Equation (10), the stress-strain relationship of the bolts must be known. The strains may easily be found for any bolt by direct proportion, assuming the lowest bolt to have a strain corresponding to the ultimate strain. Bolt forces may then be determined from the stress-strain diagram.

In practice the connection plate will have a finite stiffness and rotation will not take place about the uppermost point on the compression side. The strains will no longer follow a straight line relation, even from this new point of rotation, due to deformation not only in the connection plate, but also in the beam web. If the actual deformation of the plate were known, the strain distribution could be used to find the bolt forces and Equation (10) would still hold. If the system is economically designed, the connection plate should reach its ultimate capacity simultaneously with the ultimate condition on the bolt nearest the tension face, and at the same time the beam should have reached its full plastic moment.[3]

The assumption of a rigid plate will not serve as a design approximation since the error involved is on the unsafe side. First, the bolt forces will be less, as the true strain distribution in the bolts will be reduced due to the plate and web deformation. Second, the lever arm between C and Σ T forces will be decreased. From Fig. 7 the conditions in a connection made using a rigid plate may be compared with conditions which are assumed to exist for a plate with finite stiffness at ultimate load. The actual condition is unknown in the absence of any information concerning the distortion. This distortion is complicated by the inter-

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[3]It should be noted, however, that such a condition may not be compatible with requirements for no loss of contact at working loads.

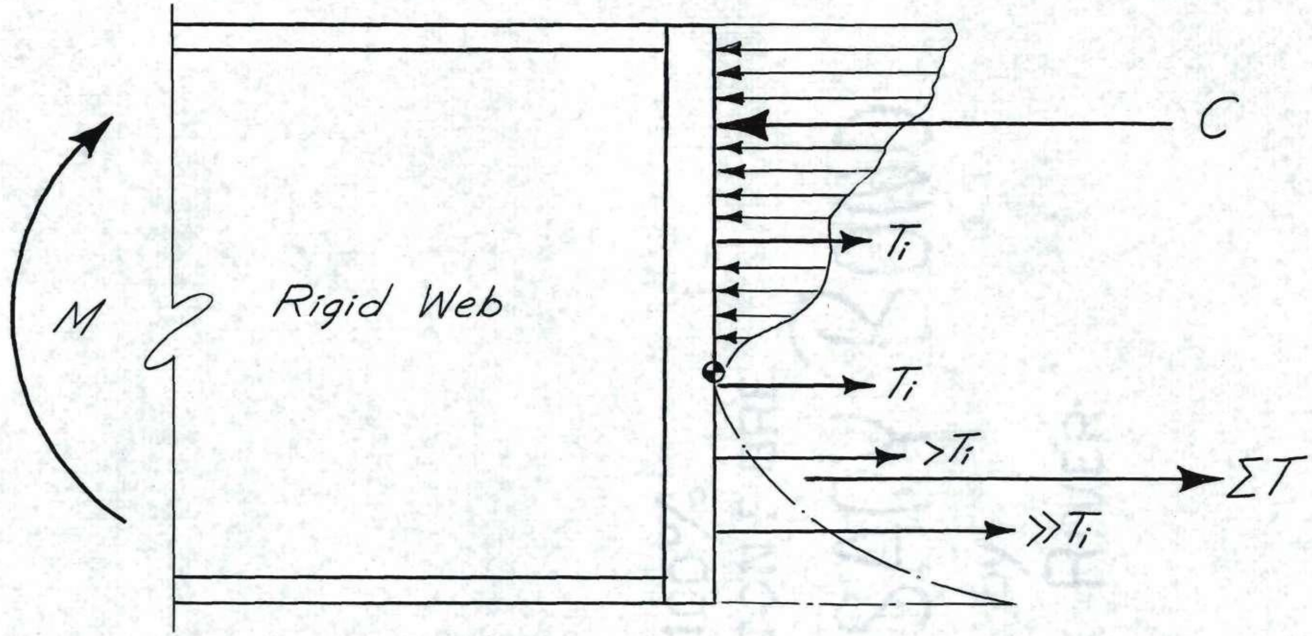


Fig. 7 Action of Direct Moment Connection with a Connection Plate Having Finite Stiffness

relation of action occurring between the bolts, connection plate, and beam. The relative stiffness of each part will determine to some extent the ultimate action of the entire connection. It is necessary to study the requirements for the connection plate before predicting the ultimate condition in the bolts.

Even at working loads, with the local disturbance of strain distribution immediately behind the connection plate, a theoretical elastic solution would be unlikely to approximate the true state of stress in the connection plate. The most rational solution for the connection plate will be an ultimate design based on plastic analysis.

From practical considerations the width of the plate should correspond to the flange width of the beam and length of the plate to the beam depth. The plate thickness is then the only variable, and must be selected to fulfill two basic requirements--sufficient strength and stiffness. It is convenient to imagine the action of a very thin plate; under load this plate would distort in two planes as shown in Fig. 8. There would be plastic regions in the plate, probably as shown in Fig. 8c. As the stiffness of the connection plate increases relative to the beam flanges and web, the distortion will begin to occur in the beam as well as in the connection plate. If enough stiffness is present, the connection plate may be expected to have very little distortion in a plane such as Section AA of Fig. 8b. Assuming a case where this distortion is negligible, the connection at ultimate load would appear as in Fig. 9. The true action undoubtedly lies between these cases. However, even rough calculations will show that the connection plate must be thicker than the beam flanges, and the latter case will therefore be used to

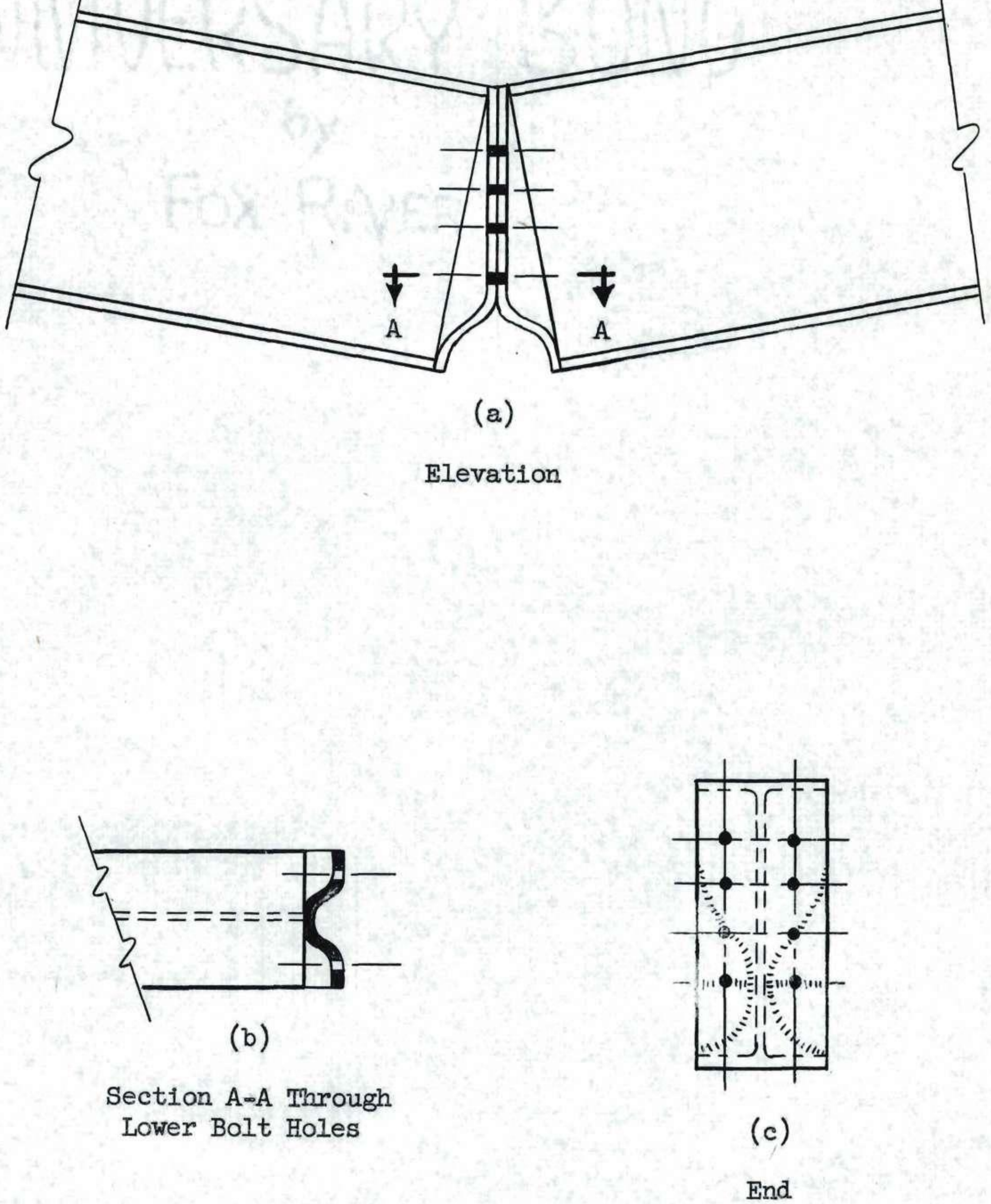


Fig. 8 Distortion of Thin Connection Plate

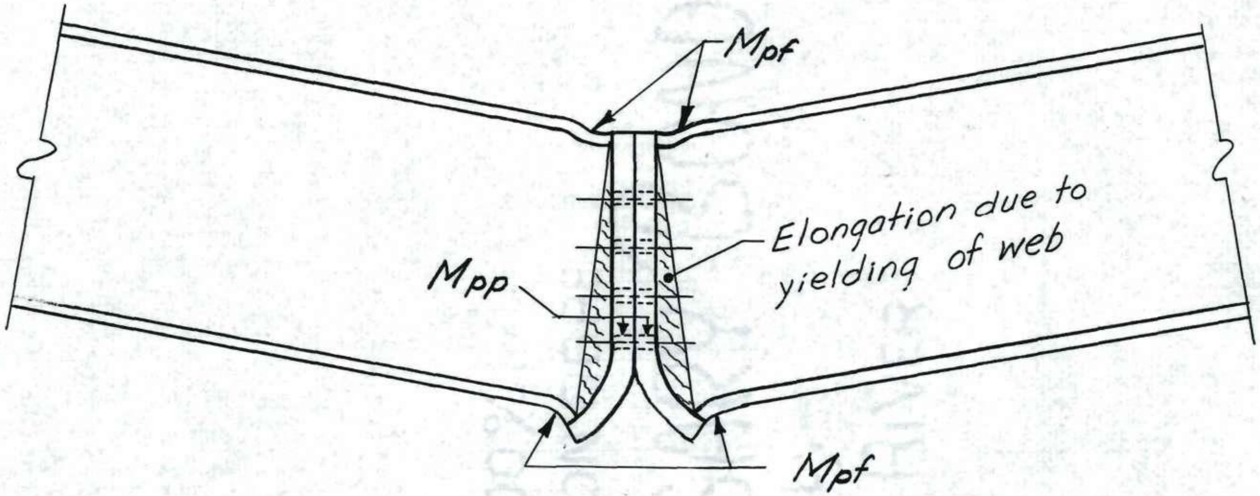


Fig. 9 Distortion of Direct Moment Connection and Beam for Practical Case

develop an expression for the plate thickness.

The derivation of the equation for the plate thickness is developed using the method of work, conventional plastic analysis (4), and assumed loading shown in Fig. 10a. The freebody of Fig. 10a is based on an assumption that a practical plate thickness will have sufficient stiffness to cause the majority of the deformation to occur in the beam web with a plastic hinge forming at the lower bolt in the connection plate itself and in the beam flanges as shown in Fig. 10b. The plastic hinges in the connection plate and flanges are assumed to form in a "T" section composed of the beam flange or connection plate and a portion of the web having a stem depth, h, such that the neutral axis lies at the intersection of the web and flange or connection plate.

From Fig. 11 the value of h may be determined if such a condition be assumed to exist.

$$\sigma_{yf} b \frac{(t_f)^2}{2} = \sigma_{yw} \frac{h^2}{2} t_w$$

$$h^2 = \frac{b(t_f)^2 \sigma_{yf}}{t_w \sigma_{yw}}$$

$$h = \sqrt{\frac{b}{t_w}} \left(\frac{t_f \sigma_{yf}}{\sigma_{yw}} \right)$$

Using a virtual displacement of the failure mechanism, expressions for the internal and external work may be written. Then from Fig. 10:

Internal Work:

$$M_{pf} \theta + M_{pf} \frac{m}{q} \theta + M_{pp} (1 + \frac{m}{q}) \theta$$

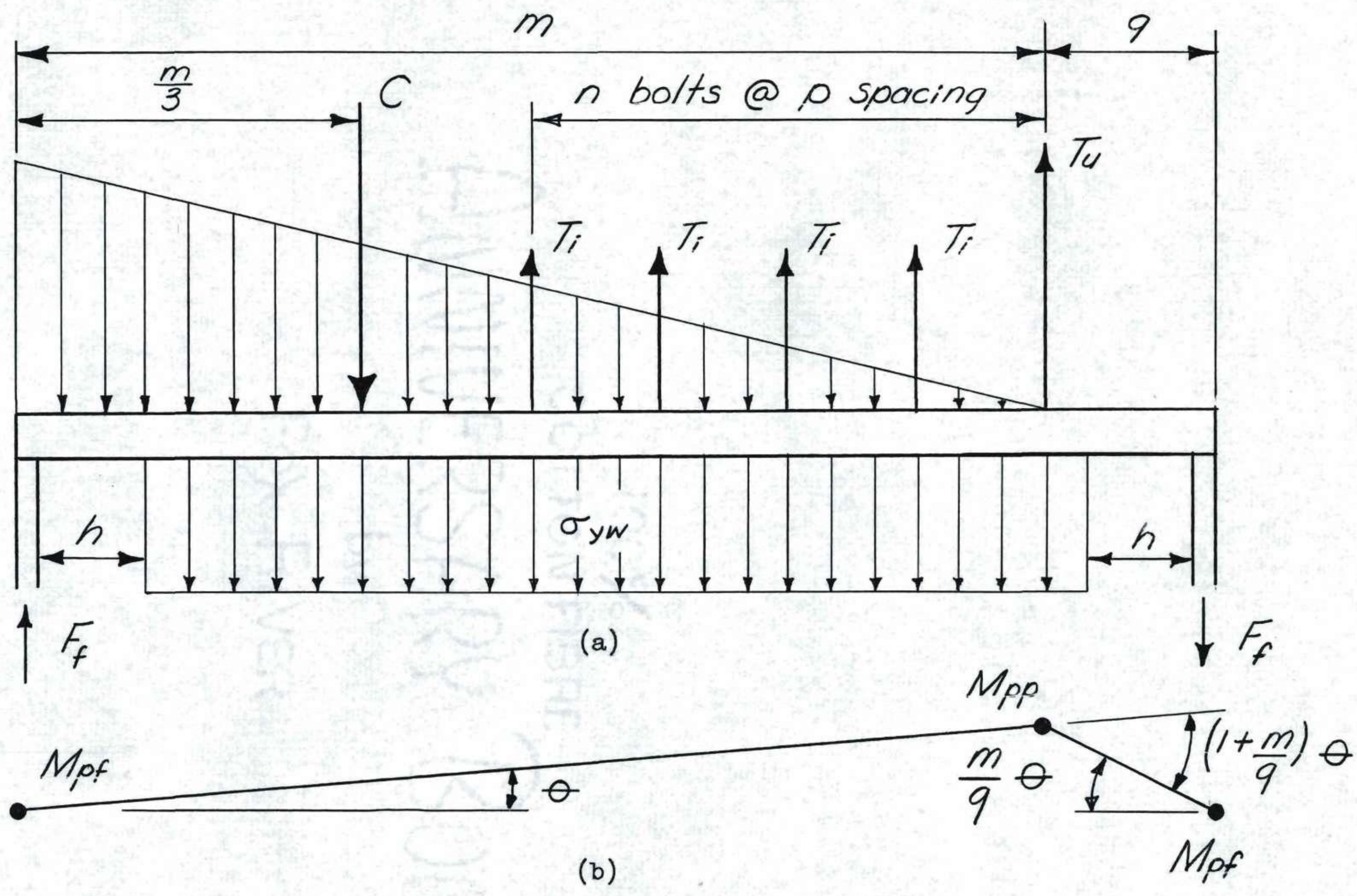


Fig. 10 Freebody and Failure Mechanism of Connection Plate

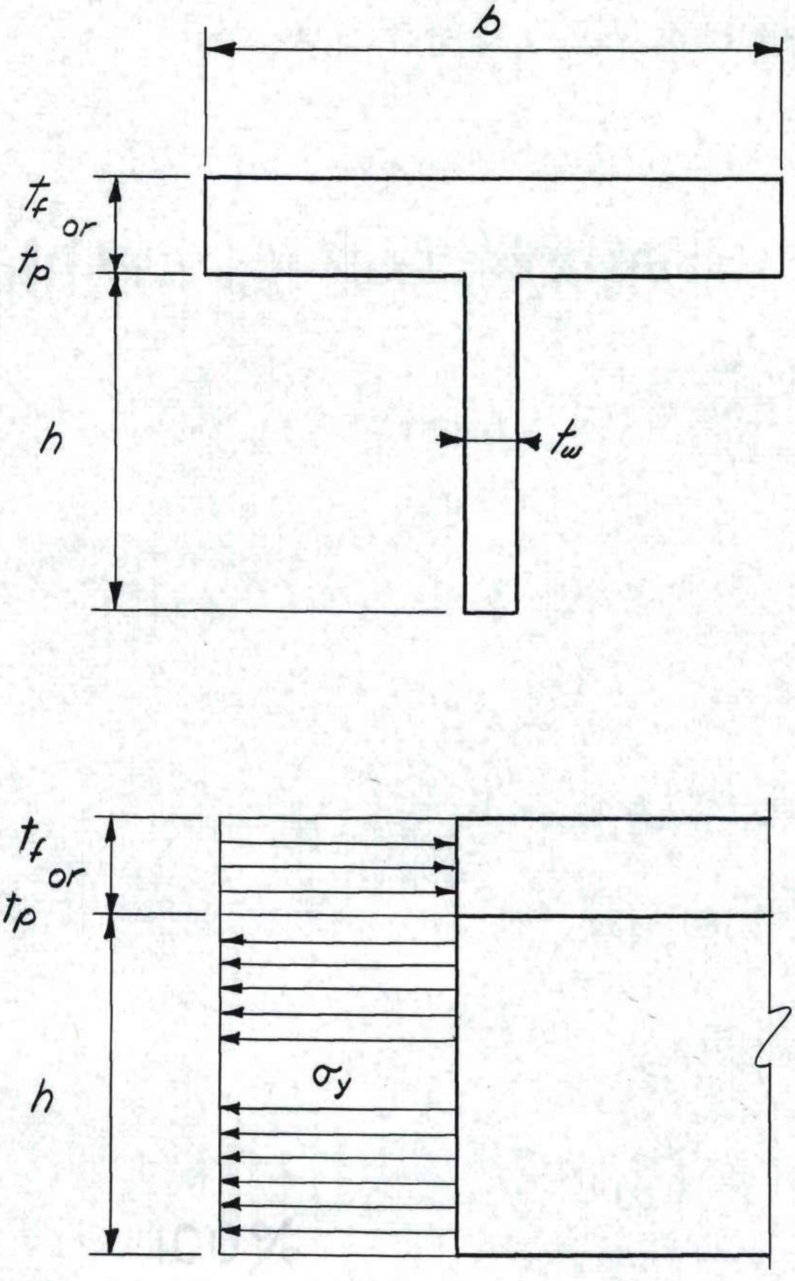


Fig. 11 Detail of Plastic Hinges for Failure Mode of Fig. 10

where $$M_{pf} = b_f(t_f)^2 \sigma_{yf}$$

$$M_{pp} = b_p(t_p)^2 \sigma_{yp}$$

External Work:

Due to bolt loads.--

$$T_u \theta m + T_i \theta (m-p) + T_i \theta (m-2p) + \ldots . T_i \theta (m-np) \text{ per Row } R$$

or $$\theta \left\{ mT_u + T_i \left[mn-(1+2+\ldots . n)p \right] \right\} R$$

Due to contact pressure.--

$$- C \frac{m}{3} \theta \quad \text{where} \quad C = \left[T_u + (n-1)T_i \right] R$$

Due to web where $h \angle q$.--

on compression side of plate hinge

$$- \sigma_{yw} t_w(m-h) \left[h + \left(\frac{m-h}{2} \right) \right] \theta$$

on tension side of plate hinge

$$- \sigma_{yw} t_w(q-h) \left[h + \left(\frac{q-h}{2} \right) \right] \frac{m}{q} \theta$$

Due to web where $h \geq q$.--*

$$- \sigma_{yw} t_w(d-2h) \frac{d}{2} \theta$$

---

*For all beams tested, $h \doteq q$. In such a case the expressions for web external work reduce to:

$$- \sigma_{yw} t_w(d-2q) \frac{d}{2} \theta$$

Since external work equals internal work:

$$(M_{pf} + M_{pp})\ (1 + \frac{m}{q}) = \left\{ mT_u + T_i \left[mn-(1 + 2 + \dots n)p \right] \right\} R -$$

$$\frac{m}{3} \left[T_u + (n-1)T_i \right] R -$$

$$\sigma_{yw} t_w \left\{ (m-h) \left[h + (\frac{m-h}{2}) \right] + (q-h) \left[h + (\frac{q-h}{2}) \right] \frac{m}{q} \right\} \tag{11a}$$

or if $h \doteq q$

$$(M_{pf} + M_{pp})(1 + \frac{m}{q}) = \left\{ mT_u + T_i \left[mn-(1 + 2 + \dots n)p \right] \right\} R -$$

$$\frac{m}{3} \left[T_u + (n-1)T_i \right] R - \sigma_{yw} t_w (d-2q) \frac{d}{2} \tag{11b}$$

where

$$M_{pf} = b_f (t_f)^2 \sigma_{yf}$$

$$M_{pp} = b_p (t_p)^2 \sigma_{yp}$$

$$h = \sqrt{\frac{b}{t_w} \left(\frac{t_f\ \sigma_{yf}}{\sigma_{yw}} \right)}$$

Since all terms are known except t_p, Equations (11a) or (11b) can be solved for the connection plate thickness. There is a small discrepancy in several of the distances involved in these equations as the exact location of the plastic hinge in the beam flange has not been specified. At most this error would change some of the distances used by an amount equal to the flange thickness. This error is small, and in view of the assumptions necessary for this derivation, not important. The resulting simplicity of the equation as derived is believed more important.

The failure mode as shown in Fig. 9 causes relatively little additional elongation in any of the bolts except the lower set. It would, of course, be desirable to have a condition closer to a rigid plate causing elongation, and thus increase tension forces in the upper bolts at ultimate load. However, the plate thickness necessary for such rigidity would probably be too large for practical application. Using Equation (11) the connection plate thickness is of the order of twice the beam flange thickness. An upper practical limit for a ratio of connection plate to flange thickness should not be much greater because of welding difficulties, remembering that the beam web is of even less thickness than the flanges.

This assumed action of the connection plate forms a basis for the evaluation of its thickness and at the same time defines the bolt forces at ultimate load. The ultimate moment to be carried is the plastic moment of the beam. Sufficient bolts must therefore be present to develop this full plastic moment when the bolts nearest the tension side have reached ultimate, with the remaining bolts at their initial tension. This means loss of contact will have occurred up to the lower bolts, and the effective moment lever arm will be less than used for the development of Equation (10). Equation (12) is developed from the freebody of Fig. 12 which is in agreement with the forces assumed acting in Fig. 10, and results in an equation similar to that of Equation (10).

$$M_b = Ca = \Sigma Ta$$

and since $\qquad \Sigma T = (T_1 + T_2 + \ldots\ldots T_n)R$

$$a = d - \left[\bar{y} + \frac{d - q}{3}\right]$$

$$\bar{y} = \frac{T_1 y_1 + T_2 y_2 + \ldots\ldots T_n y_n}{T_1 + T_2 + \ldots\ldots T_n}$$

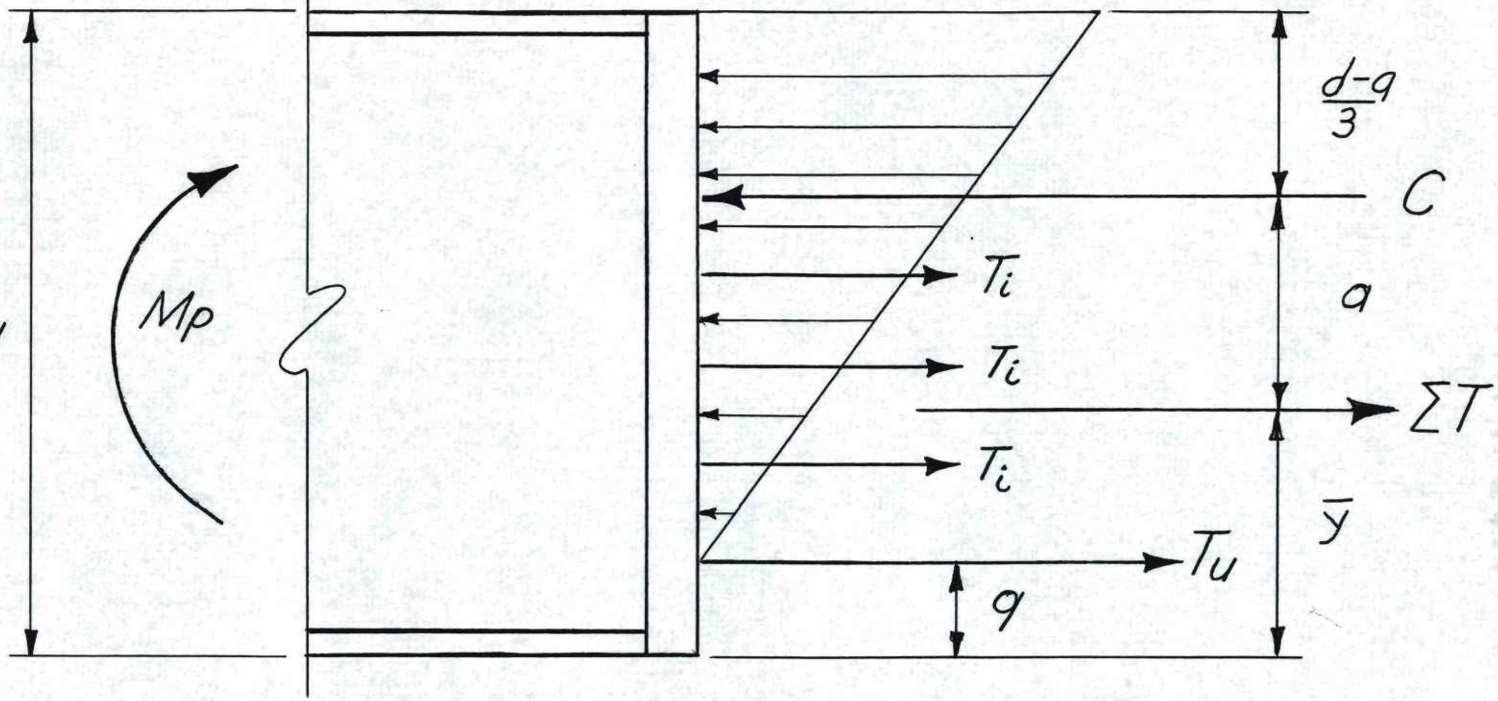


Fig. 12 Assumed Bolt Loads at Ultimate Moment

then:

$$M_b = \left\{(T_1 + T_2 + \ldots . T_n)(d- \left[\frac{T_1y_1 + T_2y_2 + \ldots . T_ny_n}{T_1 + T_2 + \ldots . T_n} + \frac{d-q}{3}\right])\right\} R$$

$$M_b = \left\{(T_1 + T_2 + \ldots . T_n)\left[\frac{2d}{3} + \frac{q}{3} - \left(\frac{T_1y_1 + T_2y_2 + \ldots . T_ny_n}{T_1 + T_2 + \ldots . T_n}\right)\right]\right\} R$$

$$M_b = \left\{\frac{2}{3}\ (d + \frac{q}{2})\left[T_1 + T_2 + \ldots . T_n\right] - (T_1y_1 + T_2y_2 + \ldots . T_ny_n)\right\} R$$

if:

$$T_1 = T_u$$

$$T_2 \ldots . T_n = T_i$$

$$y_1 = q$$

$$y_2 = q + p$$

$$y_3 = q + 2p$$

$$y_n = q + (n-1)p$$

then:

$$M_b = \left\{\frac{2}{3}\ (d + \frac{q}{2})\ \left[T_u + (n-1)T_i\right] - \left[T_uq + T_iq(n-1) + T_i\ (1 + 2 + 3 \ldots . \left|n-1\right|\ )p\right]\right\} R$$

or

$$M_b = \left\{\frac{2}{3}\ (d + \frac{q}{2})\ \left[T_u + (n-1)T_i\right]\ -T_uq - T_i\ \left[q(n-1) + \left(1 + 2 + 3 + \ldots . \left|n-1\right|\right)\ p\right]\right\} R \qquad (12)$$

and to develop the beam

$$M_b = M_p \qquad (12a)$$

Summary of Design Procedure for a Direct Moment Connection

The design of a direct moment connection may then be accomplished by selecting the number and arrangement of bolts to satisfy Equation (12) and Equation (12a), and a connection plate thickness to satisfy Equation (11). In addition, there should be a check made to insure no loss of contact at working loads from Equation (9). Shear capacity may be determined from Equation (3) or Equation (5). The factor of safety against slip under shear at working loads will normally be high enough for a connection designed to carry the full plastic moment so that no additional check is necessary; however, Equation (5) could be used to determine the ultimate shear capacity. Actually, the full plastic moment and ultimate shear capacity of the beam cannot be developed simultaneously (5). Furthermore, after slip the bolts would not be able to develop their full ultimate tensile loads (6). The situation is thus complicated if high shear exists simultaneously with the moment approaching the plastic moment, not only in the connection, but in the beam itself. Fortunately, such a situation is of rare practical importance and the design for full plastic moment with a secondary check by Equation (3) for shear at working loads will suffice.

CHAPTER III

TEST PROCEDURE AND INSTRUMENTATION

General

As stated in the introduction, this study was undertaken as a pilot program. The basic purpose of the test program was to answer the questions proposed on page 4 with a secondary objective of obtaining information to guide the conduct of a more extensive testing program. This secondary objective would naturally only be of importance if the tests indicated an affirmative answer to the basic questions. In harmony with this idea, elaborate instrumentation was not considered necessary. The test setup and procedure will be outlined before discussing the instrumentation used.

Assuming that the connection can be adequately designed from the theoretical equations, there exists only one variable which may be selected at the designer's discretion, and this is to some extent limited. This variable is the size of fasteners. Obviously, the smaller the bolt size the greater will be the number required to develop a given beam. If the size is too small there may not be sufficient space for the fasteners, while a selection of a large bolt size may lead to an impractical solution; for example, some of the smaller beams may be developed using only two large bolts. Several disadvantages of such an arrangement are apparent; probably the most important being lack of torsional stability, which in itself will preclude this as a practical arrangement.

On the other hand, both shop and field labor are increased with the smaller bolts. No specific solution can be advanced; this is one of the problems of design where the designer's judgment and evaluation of the specific situation are required. It may seem that a second variable exists once the size of the bolts has been selected, as the geometric arrangement may be varied within certain limits. But the theoretical equations actually control this also, since the most efficient arrangement would place the bolts as close to the tension flange as possible, provided there is no reversal of moment. Some beams have flanges wide enough that two rows of bolts may be used on either side of the web, enabling the designer to further increase the eccentricity in case of unidirectional moment. For members which will be subjected to moments of alternating signs the designer must choose the most efficient arrangement, varying from no bolt eccentricity for equal alternating moments to a proportional eccentricity for secondary moments of lesser magnitude.

The test program was built around the theoretical equations with the size of the fasteners and the geometry of their arrangement being the primary variables. For one series the connection plate thickness was varied. Table 1 lists the five tests performed. Series A was performed first to determine the effect of varying the connection plate thickness. Series B was designed to study effect of bolt size variation and the resulting change in geometry and eccentricity. The objective of Series C was to continue the study of performance with variation in number of bolts, using a beam having a greater depth to width ratio. The actual selection of Series A and Series B beams was based on sections available, while the Series C beam was limited by existing load transfer beams at the laboratory.

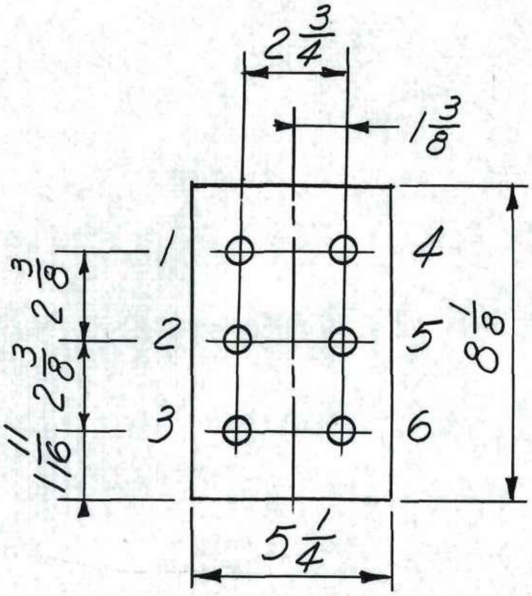
Table 1. Test Program

Series	Beam Size	Connection Plate	Bolts	Code
A	8 WF 20	3/8"	6 - 5/8" dia.	8,20-3-6,5
	8 WF 20	3/4"	6 - 5/8" dia.	8,20-6-6,5
B	8 WF 17	5/8"	6 - 5/8" dia.	8,17-5-6,5
	8 WF 17	5/8"	4 - 7/8" dia.	8,17-5-4,7
C	18 WF 50	1"	10 - 7/8" dia.	18,50-8-10,7

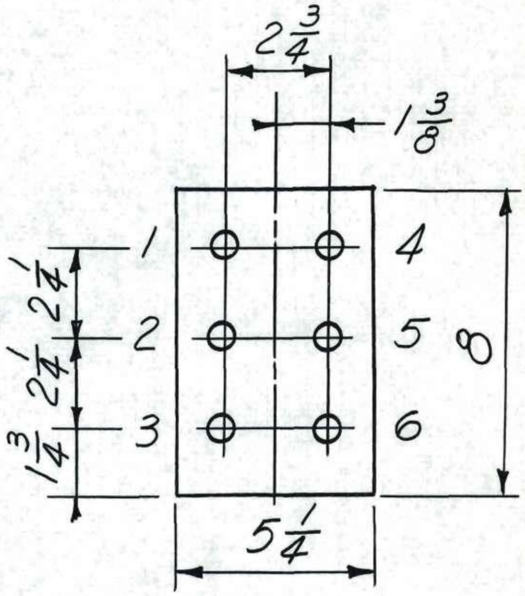
Figure 13 is a detailed drawing of the connection plate used for each test and also shows the numbering of bolts for later reference. Future reference to the individual test will be made by a code which is also listed in Table 1. This code is: the beam size, and weight--the connection plate thickness in eighths of an inch--and the number of bolts, followed by the bolt diameter in eighths of an inch.

Specimen Preparation

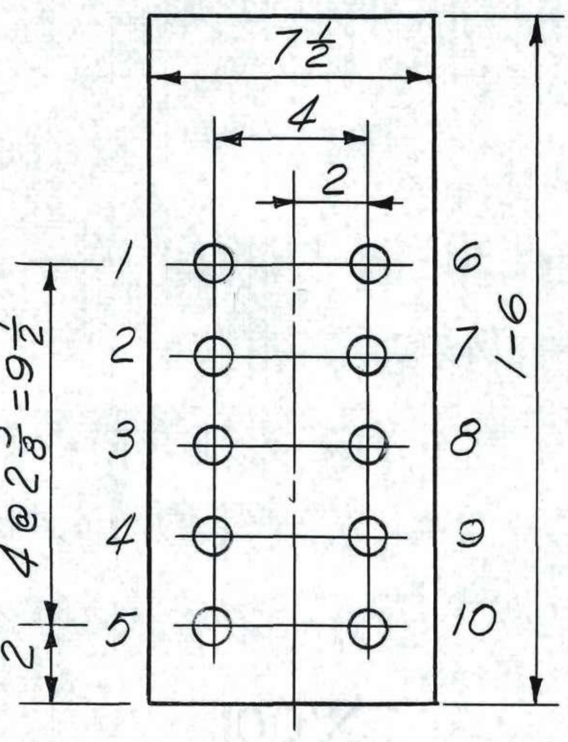
All specimens were prepared in the shop of the School of Civil Engineering, Georgia Institute of Technology. No special laboratory technique was exercised in the preparation of the test specimens; rather, they were fabricated in a manner which represented the usual care and tolerance of a first grade structural fabrication shop. Holes were drilled for each test simultaneously in both plates. All beam flanges were chamfered at 45 degrees prior to welding, using a portable grinder. Welding of all specimens except 8,17-5-6,5 was done by the writer using 6012 electrodes. Specimen 8,17-5-6,5 was prepared by a commercial



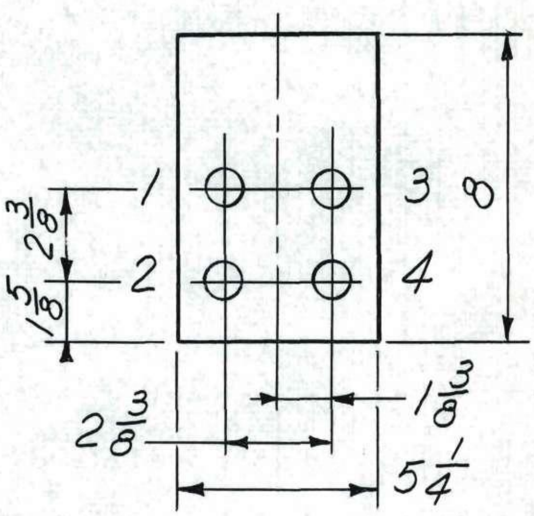
Tests 8,20-3-6,5
8,20-6-6,5



Test 8,17-5-6,5



Test 18,50-8-10,7



Test 8,17-5-4,7

Note: Drill all holes:
For $\frac{5}{8}$ ϕ bolts, $\frac{11}{16}$
For $\frac{7}{8}$ ϕ bolts, $\frac{15}{16}$

Fig. 13 Connection Plate Details

welder. This was considered advisable since the writer was an inexperienced welder and cooling cracks developed in the web welds of specimens 8,20-3-6,5 and 8,20-6-6,5. These will be discussed in detail in Chapter IV. The remaining specimens were subsequently welded by the writer with no difficulties.

Testing Procedure

All specimens were loaded and instrumented in a similar fashion; a detailed explanation of test 18,50-8-10,7 will be given. Figure 14 is of this test, but is typical of all tests. Table 2 gives dimensions used for the individual tests; the letters refer to Fig. 15.

Table 2. Dimensions of Test Setup

Test *	a	b	c	d	e
8,20-3-6,5	34	54	142	8.5	3
8,20-6-6,5	30	54	138	8.5	3
8,17-5-6,5	26	24	74	8.5	3
8,17-5-4,7	20	24	68	8.5	3
18,50-8-10,7	34	30	94	19.5	4

*All dimensions in inches

Specimens were loaded in a RIEHLE Model PS-450, screw powered, constant strain testing machine with a capacity of 450,000 pounds. Load was transferred directly from the universal head of the testing machine, through a reinforced 8 WF 67 loading beam, to supports consisting of a 5 3/4-inch by 4 5/8-inch by 2-inch steel block with a 1 1/8-inch diameter by 5-inch rod centered and welded to this block. The supporting blocks were placed so that the 1 1/8-inch diameter rod bore on the test specimen centered

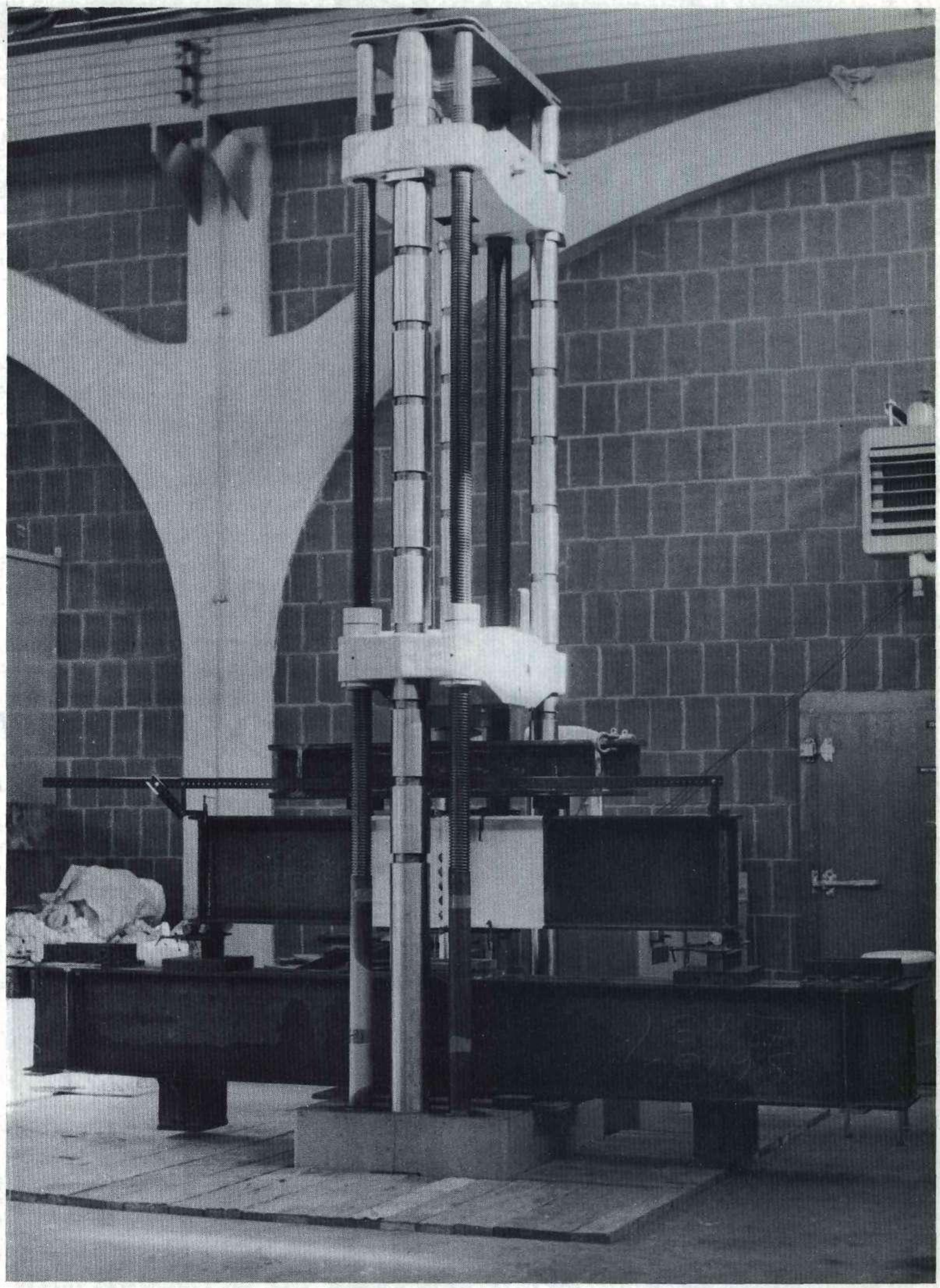
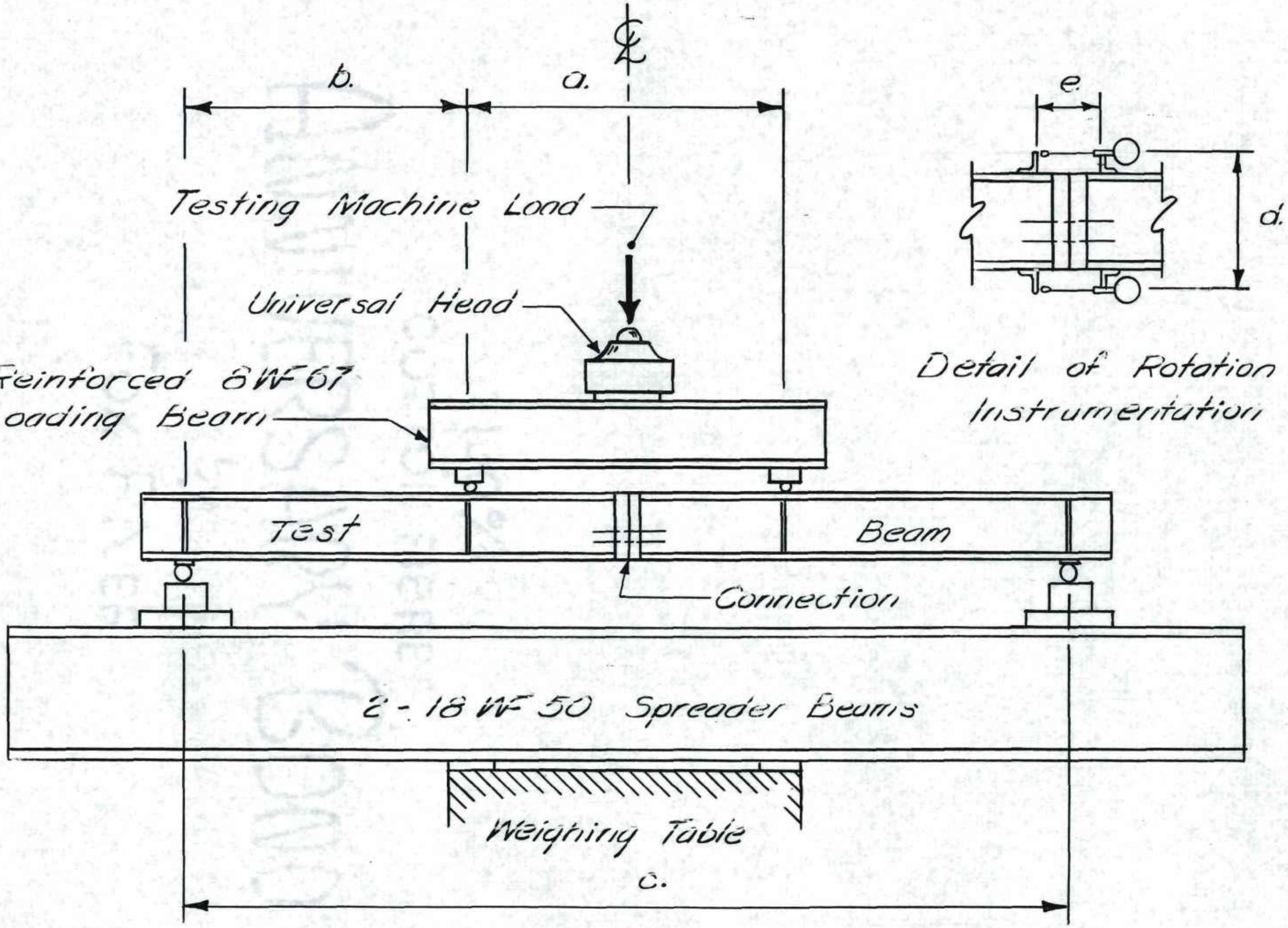


Fig. 14 Typical Test Arrangement



See Table 2 for Specific Dimensions

Fig. 15 Testing Arrangement

approximately at the one-third points. The connection was thus loaded in pure moment. In test 18,50-8-10,7 the points of load application were spaced approximately twice the beam depth; for all other tests the spacing was at least two and one-half times the beam depth. Reaction supports for the test specimens were 2-inch diameter rollers, supported on steel blocking plates which rested on two 18 WF 50 load transfer or spreader beams. Stiffeners were provided at points of load application and at the reactions of the test specimens. The spreader beams transferred the reactions to the weighing table of the testing machine. Specimens were loaded in predetermined increments modified by action of the specimen. The rate of loading was varied according to the stiffness of the specimen, so that a visual check on the loading rate, obtained by observation of the testing machine pointer, was not excessive. In no case was the specimen loaded at a rate greater than one-tenth inch per minute free head travel.

Instrumentation

Instrumentation was provided to measure rotation of the connection and centerline deflection of the specimen. Rotation measurements were obtained by clamping two micro-mechanical dial gages to the north side of the connection on both the tension and compression flanges, with the gages bearing on angles clamped to the beam flanges on the south side of the connection. Rotation instrumentation thus consists of a gage on the tension flange and its companion gage on the compression side. Readings from one such pair of gages gave rotation of the west side of the connection and from the other pair rotation of the east side. The average of the west and east rotations has been used for discussion. Centerline deflection was obtained from a fifth micro-mechanical dial gage bearing under the center of the connection. This deflection dial was supported by a magnetic stand resting on the spreader beams for the Series A and B tests. Since deflection of the spreader beams would also appear in the dial readings, a truss was constructed to support the deflection dial for the 18,50-8-10,7 test. The truss was clamped to the test specimen directly over the supports, and the deflection dial was clamped to the truss to bear on the center of the connection.

Bolt elongations were obtained by a machinist's micrometer equipped with a ratchet handle and special tips. These conical shaped tips fit drilled and countersunk holes in the head and tail of each bolt.

Initial pretension of the bolts was also controlled by the elongation read with the micrometer. After the first tightening all bolts were checked and retightened, if necessary, to the desired elongation. Some difficulty was experienced in obtaining consistent elongation in all bolts. This was more noticeable in the thinner plates and was due to the plates not fitting together perfectly. The elastic action of the plates would change previously tightened bolts during the tightening of subsequent bolts. Actually, very little variation was present in the plates, but a few thousandths of an inch movement would appreciably effect the bolt stress. From a practical standpoint this is of little consequence as all that is required to assure sufficient bolt stress is retightening, but exact control is difficult to obtain.

Figure 16 shows the rotation dials on the west side of the beam, the deflection dial, and the conical bolt holes.

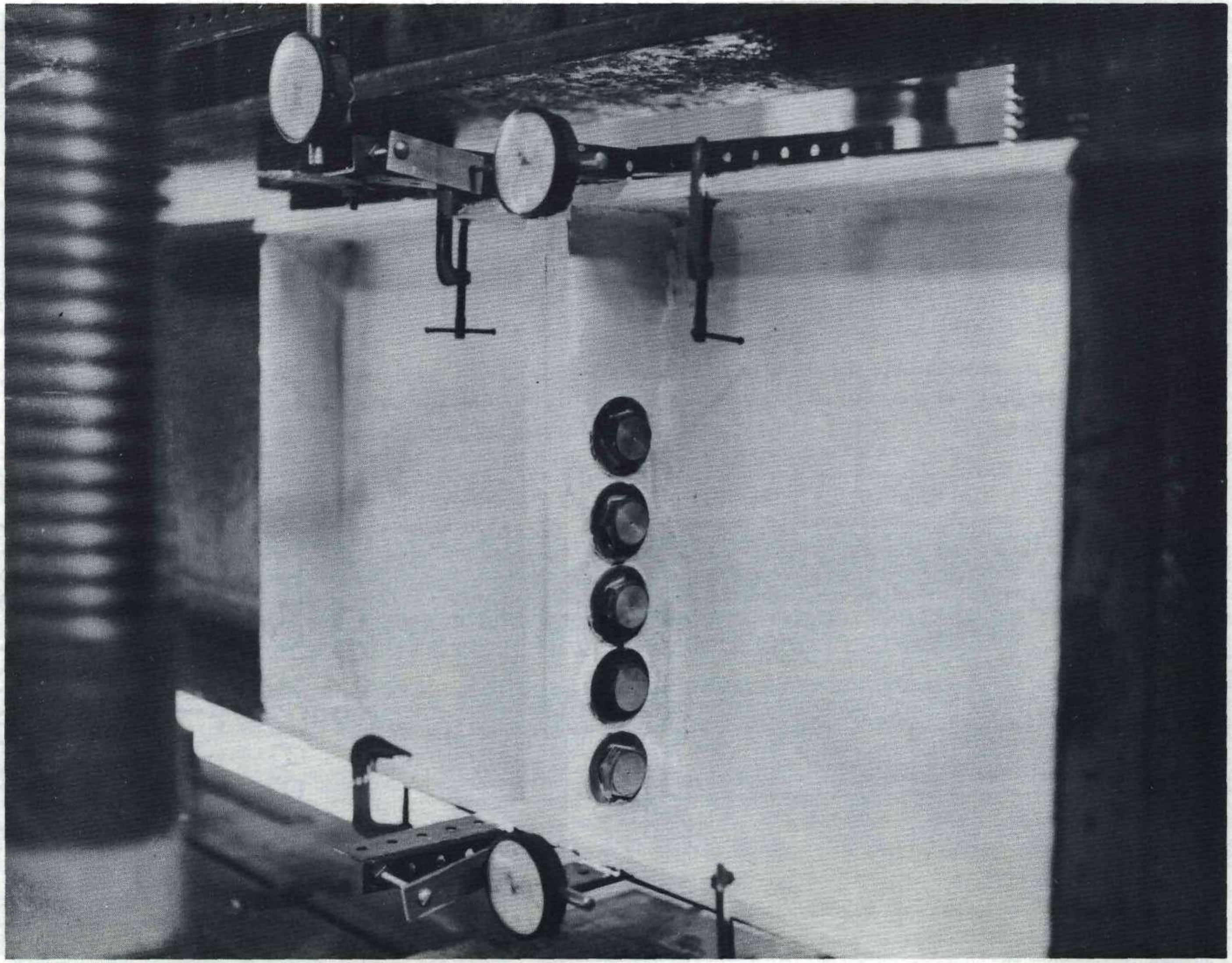


Fig. 16 Instrumentation for Typical Test

CHAPTER IV

TEST RESULTS

General

Each Series and each test in a Series will be discussed individually; then certain correlations between tests will be discussed. In the case of an individual test, the pertinent points considered are: 1) moment developed, 2) rotation capacity, 3) deflection, and 4) failure mode. Conclusions are then drawn. Correlation of individual tests will show the effect of: 1) increased connection plate thickness, and 2) increased bolt size.

Series A, General

The A Series consisted of two tests: 1) an 8 WF 20 beam with a 3/8-inch plate, and 2) an 8 WF 20 beam with a 3/4-inch plate. Six 5/8-inch bolts concentrically spaced were used for both tests. By the theoretical equation for the connection plate design, Equation (11), a 9/16-inch plate is required to develop the six 5/8-inch bolts. The six 5/8-inch bolts will theoretically develop 284 kip-inches as found from Equation (12), while the beam plastic moment is 628 kip-inches. If the theoretical equations are correct, test 8,20-3-6,5 should fail in the connection plate and test 8,20-6-6,5 should fail in the bolts without damage to the connection plate. It was proposed that should this sequence develop, the six 5/8-inch bolts in test 8,20-6-6,5 would be replaced by four 7/8-inch bolts. This arrangement, a 3/4-inch plate and four 7/8-inch bolts, would theoretically develop the full M_p. Unfortunately, this third test of Series A could not be run, as the specimen failed initially through the web fillet weld with subsequent damage to the connection plate. In fact, both specimens of Series A failed in this manner. This failure was not due to faulty design nor is there any indication that the web fillet weld is critical, if properly welded. Close examination revealed that in both tests there were internal cooling cracks in the welds which were easily distinguished from the ultimate stress failure. As mentioned in Chapter III, this welding was done by the writer, an inexperienced welder, and the fault lay in insufficient amperage during the welding. No difficulty was experienced with the welds in the B and C Series; and as indicated by Figs. 23, 24, and 31, considerable deformation occurred in these welds with no failure.

Series A, Test 8,20-3-6,5

Even though both tests of Series A experienced a sudden weld failure, it is believed the failure in test 8,20-3-6,5 occurred after the ultimate strength was reached. This is indicated by the rotation curve, Fig. 17a. Discussion will be based on this assumption.

Moment developed.--Based on a yield strength of 33 kips per square inch, the M_p of an 8 WF 20 beam is 628 kip-inches. From Equation (12), using the minimum elastic proof load of 17.3 kips and an ultimate strength of 27.1 kips, the six 5/8-inch bolts will theoretically develop 284 kip-inches, or 45 per cent M_p. Bolt loads, obtained from elongation readings just prior to failure, and Equation (12) indicated a moment of 282 kip-inches or 45 per cent M_p. The actual test moment as computed from the testing machine load was 44 per cent M_p.

Rotation.--Figure 17a shows a plot of the connection rotation versus the moment. The measured connection rotation has been divided by the theoretical beam rotation that would be present when the outermost fibers reached a theoretical yield of 33 kips per square inch. Also plotted is the theoretical rotation for an 8 WF 20 beam under the same loading conditions. The connection ultimate rotation was 22 times larger than the theoretical yield rotation of the beam.

Deflection.--Figure 17b is a plot of the load versus deflection; also shown is the theoretical deflection curve for the beam. In view of the fact that the entire connection, both bolts and connection plate, is considerably weaker than would be required to develop the beam, this curve has value primarily as a qualitative comparison with other tests.

Failure mode.--The failure of this test occurred suddenly in the web welds at 44 per cent M_p. Definite yielding of the connection plate on a horizontal line through the lower holes had previously occurred at 39 per cent M_p. There was also yielding on a vertical line through the holes which was visible on the face of the plates only after disassembly. The distortion in this plane was, however, less than the distortion along the horizontal line and the latter is considered the primary failure, even in this thin plate. No visible damage was observed in any of the bolts.

Conclusions.--The entire results of this test are clouded by the manner of failure. It is impossible to say how much effect the initial cracks in the weld had on the test. These cracks could be expected to increase both the rotation and deflection. Possibly they also lowered the load at which the connection plate yielded, although it is not believed this

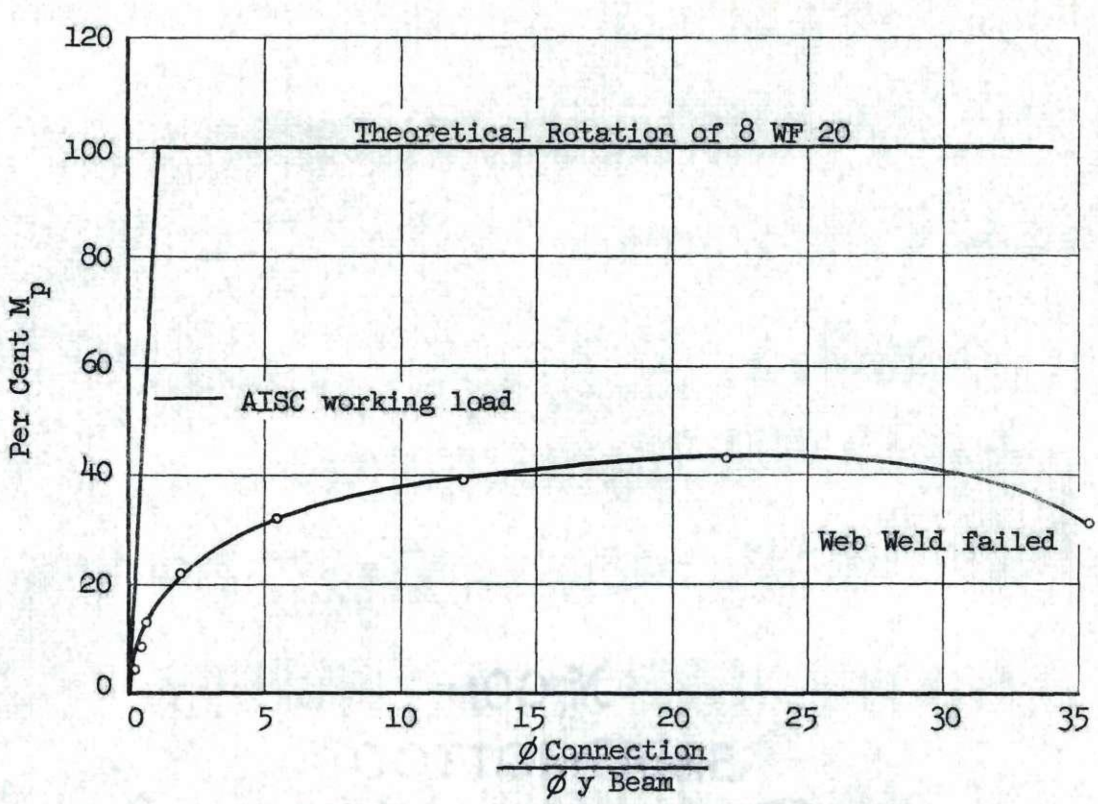


Fig. 17a Moment-Rotation Test 8,20-3-6,5

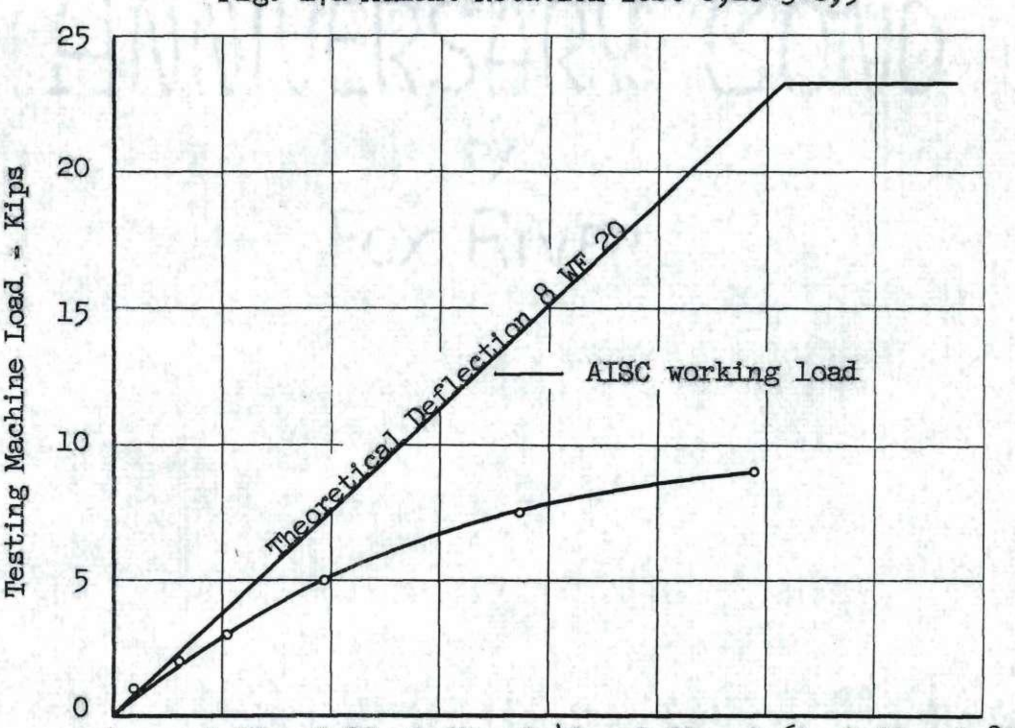


Fig. 17b Load-Deflection Test 8,20-3-6,5

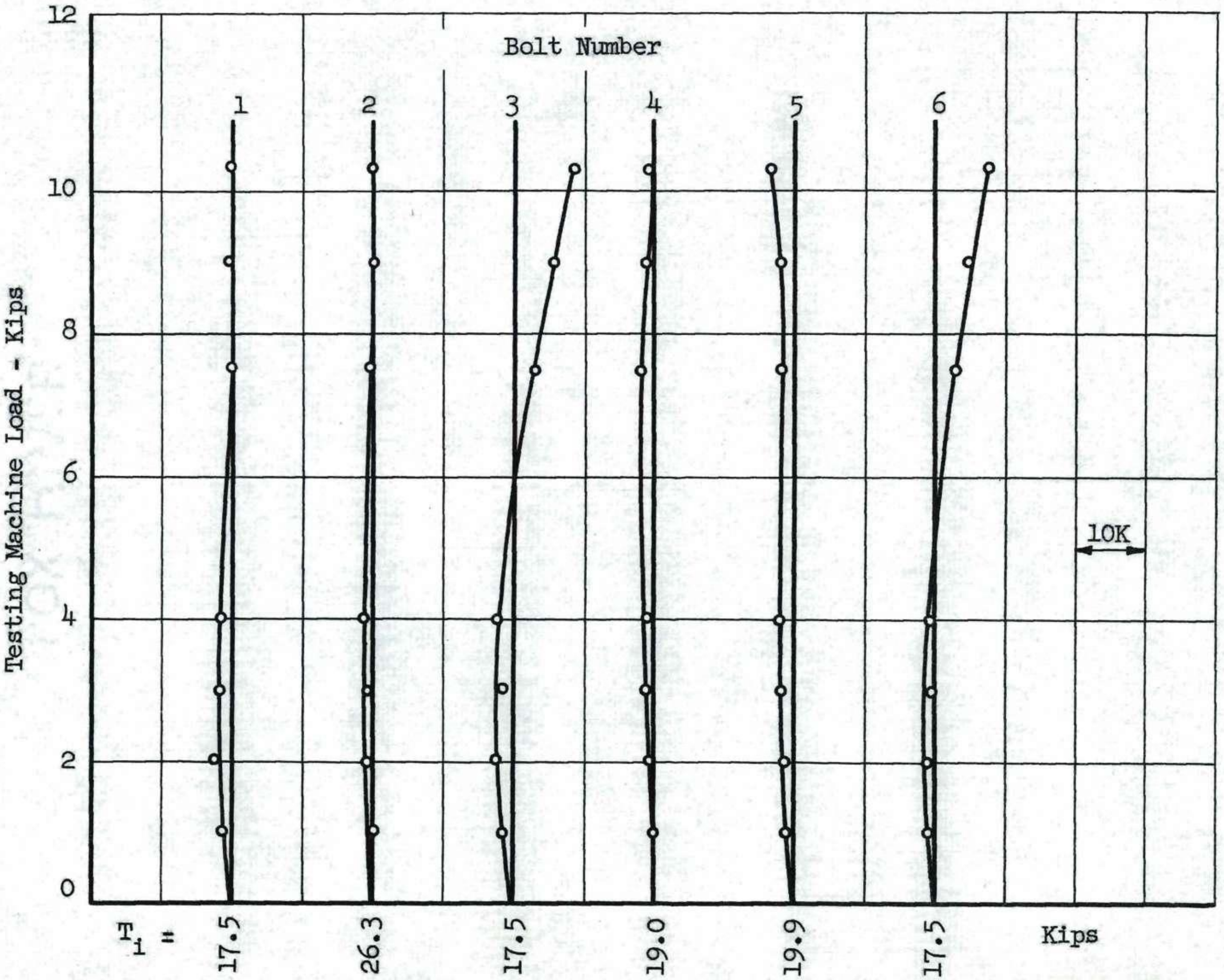


Fig. 18 Bolt Forces - Test 8,20-3-6,5

effect was great, since the initial weld cracks were only on one side of the web near the center and the plate yielded through the lower set of holes. The bolts did not reach their ultimate capacity; but results of later tests indicate that, even after the yielding of the plate, increased deformation can cause the ultimate failure of the bolts, at little increase in load capacity, however. For these reasons it is believed that very little additional load could have been carried by this test, even with good welds, if deformed to bolt failure. The bolts are stronger than minimum requirements (See Fig. 35a) and the connection carried the theoretical ultimate load based on Equation (12) using minimum bolt tensions, with no actual damage to the bolts. The close check of test load and calculated load obtained from the bolt readings just prior to the weld failure indicates the validity of Equation (12). Although the deflection and rotation curves are functions of both the plate thickness and bolt characteristics, the lack of stiffness in this test is primarily due to the undersize plate as the bolt elongations during testing were small. As previously stated, a 9/16-inch plate would theoretically be required to develop 45 per cent M_p. The fact that a plate of two-thirds this thickness developed 44 per cent M_p may be due to several reasons. Although no specimens were cut from these plates, it is reasonable to expect a yield strength of greater than 33 kips per square inch from a thin plate; but more important than this, after strain hardening the plate could be expected to develop strength considerably higher than yield strength. Also, the restraint furnished by the beam and bolts would cause the ultimate failure to involve action including not only moment but direct tensile forces similar to membrane action, particularly for a thin plate. For this preliminary study there was no attempt made to develop equations for theoretical capacity of plates of thickness less than required to develop the beam. Such plates are of no practical importance as the deflections would control the design rather than the strength; further discussion of this will be found on page 78.

The real significance of this test is the indication that an undersize plate will not result in a sudden failure, but only in excessive rotation and deflection, and that the predicted bolt action is not influenced by reasonable variation in plate thickness.

Series A, Test 8,20-6-6,5

The plot of moment versus rotation for this test shows obvious premature failure. In fact, it is believed that the entire results are invalidated by the weld cracks present before testing. Cracks opened and were visible at approximately 20 per cent M_p. In view of this, detailed quantitative results of this test are not presented, although a plot of rotation, Fig. 19a, and of deflection, Fig. 19b, is included. Nevertheless, the beam developed 62 per cent M_p. It would be mere speculation to say what results would have been obtained had the welds been properly made. However, the overall results were sufficiently greater than test 8,20-3-6,5 so as to substantially support the connection plate equation, Equation (11).

Series B, General

Although the results of Series A were confused by the weld failures, all indications were that the basic equations for design, Equation (11) and Equation (12) were correct. Series B was thus designed accordingly,

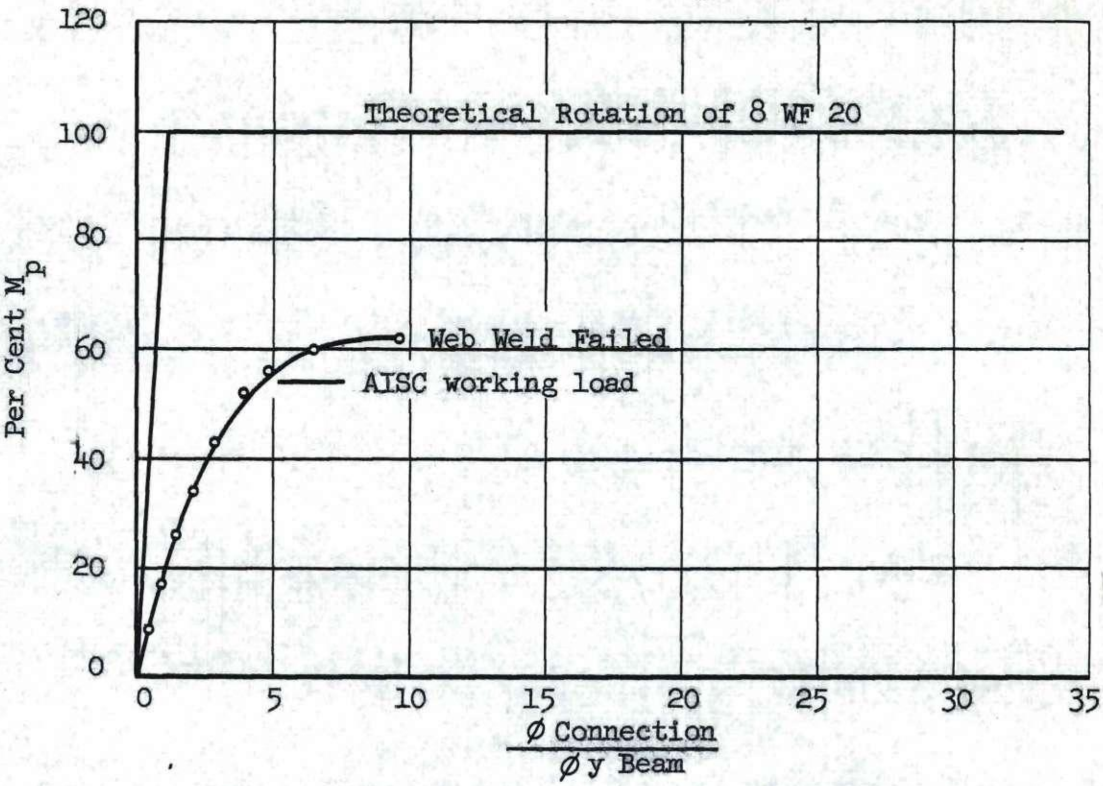


Fig. 19a Moment-Rotation Test 8,20-6-6,5

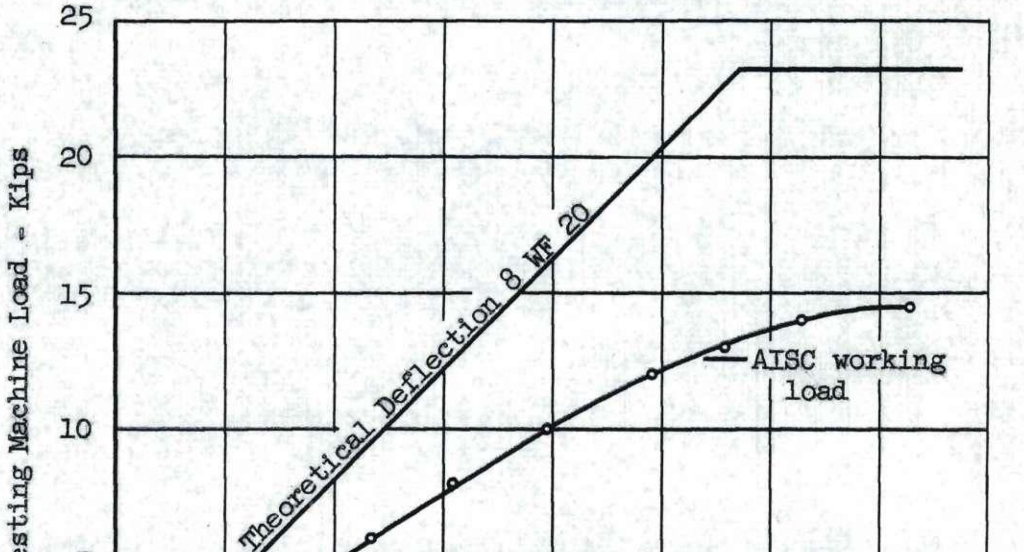


Fig. 19b Load-Deflection Test 8,20-6-6,5

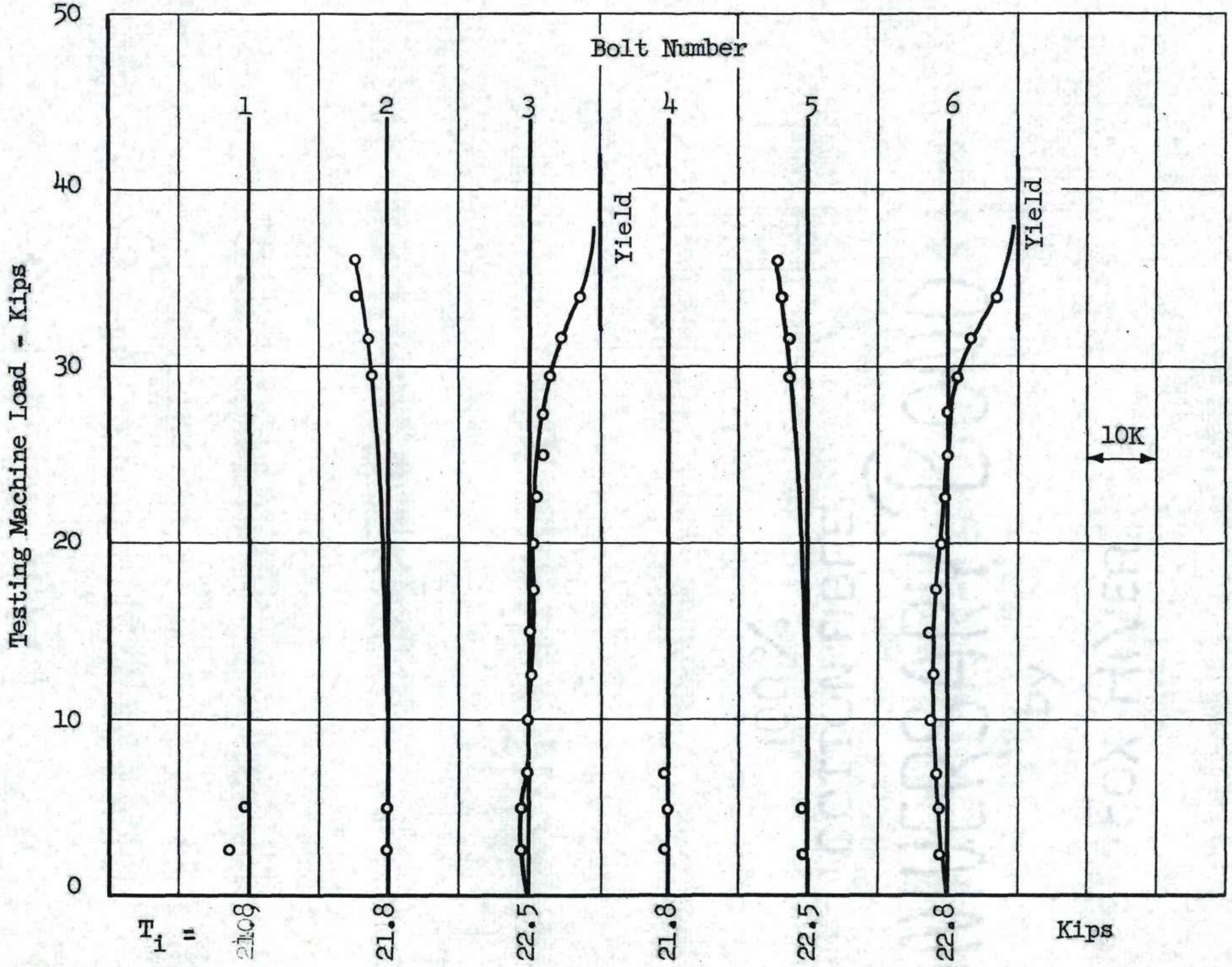


Fig. 20 Bolt Forces - Test 8,17-5-6,5

and consisted of two tests to study the effect of bolt size variation: 1) an 8 WF 17 beam with six 5/8-inch bolts concentrically spaced, and 2) an 8 WF 17 beam with four 7/8-inch bolts eccentrically located. By the theoretical equation for the connection plate design, Equation (11), a 9/16-inch plate is also required to develop the six 5/8-inch bolts used with this beam, while a 3/4-inch plate is required to fully develop the four 7/8-inch bolts. Five-eighths inch plates, an average of the two theoretical thicknesses, were used for both tests so that the difference in results would be due only to the bolt size. From the theoretical equations, test 8,17-5-6,5 should fail to develop M_p, while test 8,17-5-4,7 should develop M_p.

Series B, Test 8,17-5-6,5

Moment developed.-- Based on a yield strength of 33 kips per square inch, the M_p of an 8 WF 17 beam is 521 kip-inches. From Equation (12), using the minimum elastic proof load of 17.3 kips and an ultimate strength of 27.1 kips, six 5/8-inch bolts will theoretically develop 280 kip-inches or 54 per cent M_p. Due to difficulty in torquing the bolts of previous tests to an exact value, the desired elongations were increased for this test such that the initial bolt tensions would be 22 kips. Twenty-two kips is the average value of the theoretical minimum elastic proof load and the theoretical minimum ultimate strength. The desired value was obtained with more success in this test as shown in Fig. 20. The actual test moment as computed from the testing machine load was 433 kip-inches or 83 per cent M_p. The moment obtained from substitution of the bolt loads at the ultimate test load in Equation (12) is 326 kip-inches or 63 per cent M_p. This gives a poor check when compared with the moment of 82 per cent M_p obtained from the testing machine load. The reason for this discrepancy was difficult to interpret at the time of the test. Two possibilities exist: 1) experimental error, and 2) inapplicability of Equation (12). Interpretation was further complicated by the weld failure of the previous tests, and the lack of knowledge as to the reliability of the results obtained. The tests were continued assuming Equation (12) to be applicable, and in retrospect, the 19 per cent error of test 8,17-5-6,5 seems best attributed to several sources of experimental errors which will be discussed in the conclusions.

Rotation.--Figure 21a is a plot of the connection rotation versus the moment. As in all tests, the rotation plotted is the ratio of the connection rotation to the theoretical beam rotation at yield. The connection ultimate rotation was 34 times larger than the theoretical yield rotation of the beam. The moment-rotation relation was linear to approximately 30 per cent M_p.

Deflection.--Figure 21b is a plot of the load versus deflection; also shown is the theoretical deflection curve for the beam. The ultimate deflection of the beam with the direct moment connection is 3.8 times greater than the yield deflection for the beam. At a stress of 20 kips per square inch, the deflection ratio is 1.65.

Failure mode.--The ultimate failure of this test occurred in bolt No. 6 as expected, although at a load of 36.1 kips, or 83 per cent M_p, considerably in excess of the predicted load based on minimum bolt strength. The nut on bolt No. 3 failed on reloading at 35.5 kips. The bolt failure was actually a stripping of the threads in the nuts with some resulting

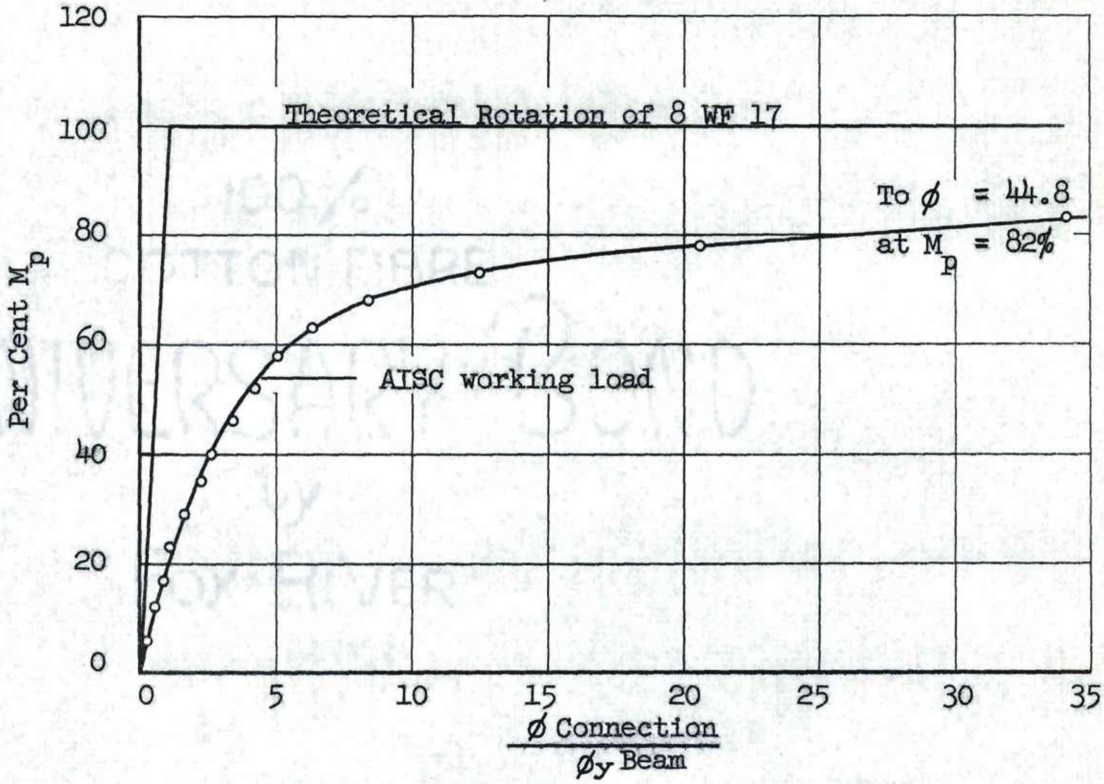


Fig. 21a Moment-Rotation Test 8,17-5-6,5

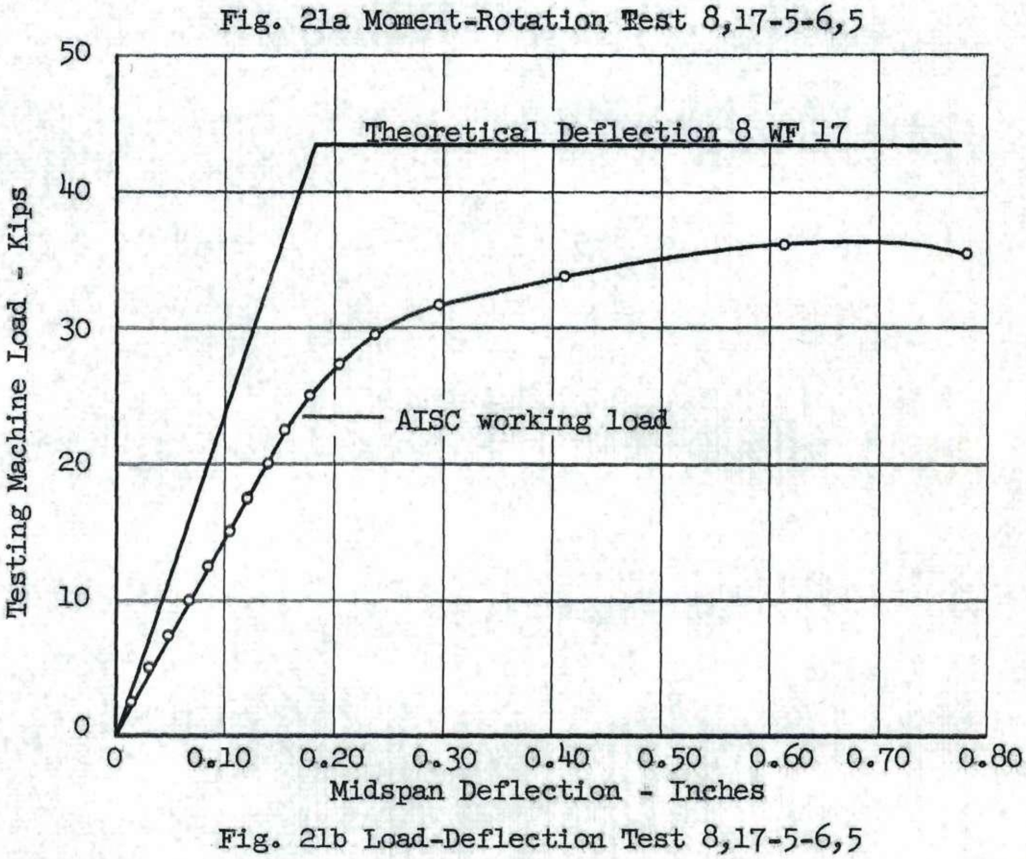


Fig. 21b Load-Deflection Test 8,17-5-6,5

damage to the bolt threads. Yielding of the plate occurred on a horizontal line through bolt hole Nos. 3 and 6 and at a load of 30.3 kips or 69 per cent M_p. On disassembly there was also evidence of connection plate yielding on a vertical line through the bolt holes. Even after initial slip of the nuts, the beam held a load that wavered around 30 kips under increasing deformation sufficient to cause local buckling of the compression flanges.

Conclusions.--Although certain discrepancies between the theoretical equations and the test results exist, it is believed that they may be explained in a manner that does not invalidate the equations. The ultimate failure load of 83 per cent M_p was in reality not unexpected as the predicted load was based on minimum required bolt strength and the bolts were known to have much higher strength. Some doubt does exist as to exactly how strong the bolts were. The test curve shown in Fig. 35a is the result of plotting the average values of two bolts. Both test bolts experienced tensile failure, but in the case of the bolts in the beam test, the failure was by stripping of the nut. The elongation of bolt No. 6 in the beam test was almost exactly the average elongation of the test bolts, but bolt No. 3 in the beam test experienced an apparent elongation 34 per cent greater than the average elongation of the test bolt. At the time of the test it was believed possible that bolt No. 3 did elongate as indicated by the data and had strength greater than obtained from the test bolts. Such action alone could not, however, explain the full 19 per cent error as a force of 67 kips in bolt No. 3 would have been necessary to obtain the required 83 per cent M_p from Equation (12). A value of 67 kips is highly unlikely for a 5/8-inch bolt. After a review of all test data, several sources of error in obtaining bolt elongations are apparent: 1) dirt and trash on the micrometer tips or in the conical bolt holes, 2) body heat expanding the micrometer during prolonged use, and 3) bending present in the bolts near ultimate beam load values.

The first of these possible errors is obvious, was recognized before running any tests, and care was taken during all testing to avoid dirt or trash on the micrometer or in the bolt holes.

The second source of error was discovered during a later group of tests; however, the temperature variations at the time were extreme. It is not believed that this source of error is important for the tests of this study as the laboratory was approximately 75° F. during all testing and the micrometer was held in the hand for only three or four minutes at a time while taking readings.

The third source of error is actually believed to account for the apparent large elongation in bolt No. 3, as this bolt was bent at the ultimate load value. This bending would cause an apparent increase in elongation due to the misalignment of the micrometer tips in the conical bolt hole. Thus it is believed that bolt No. 3 failed at an elongation and load similar to the test bolts, although the bending may also have caused a wedging action on the nuts with a correspondingly higher stripping load. However, some other explanation must be sought for the 19 per cent error as it is seen to be unlikely that bolt No. 3 was stressed to a value much in excess of the ultimate test bolt values. After review of data from all tests, Equation (12) is considered to be essentially correct.[4] The large error of this test seems to be best explained by a

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[4]See page 79 for further discussion.

drop off in load of bolt Nos. 1 and 4 as the moment approached ultimate.

The effect of such a situation on Equation (12) was overlooked at the time of testing and elongation of these bolts was not monitored during the latter portions of the test. A decrease in elongation of approximately 0.003 of an inch would have dropped these bolt loads to a value around 10 kips. With such values, and indicated values for all other bolts, Equation (12) will check the test moment. A decrease of 0.003 of an inch for these bolts is certainly possible, especially when the magnitude of decrease in bolt No. 2 is observed. Of course, this explanation is dependent on the correctness of Equation (12), but in view of results obtained in other tests, there would not appear to be any gross error in Equation (12) and the above explanation is further confirmed by test 8,17-5-4,7.

As in test 8,20-3-6,5 the connection plate proved stronger than predicted, and the same explanation of higher than minimum yield strength, strain hardening, and some membrane action seems a reasonable explanation. Quantitative discussion of rotation and deflection results for this test are not highly important, as the connection was underdesigned, although actually carrying 83 per cent M_p. Possibly, the most important observation is that rotation and deflection are approaching the beam theoretical value as the connection plate thickness and bolt size approach the theoretical.

Series B, Test 8,17-5-4,7

This is the first test that was designed to develop the full plastic moment of the beam; however, a plate thickness of 5/8-inch was used in place of the computed 3/4-inch plate. As previously mentioned, this was an average thickness of the required thickness for the two Series B beams, and was used so that variation of results in this Series would be solely a function of bolt size. It was not expected that the lesser plate thickness would prevent test 8,17-5-4,7 from reaching its ultimate capacity as the computations for the plate thickness are based on minimum strength, and test 8,17-5-6,5 indicated higher strength for the particular 5/8-inch plates used.

Moment developed.--The test load was 521 kip-inches or 100 per cent M_p. Four 7/8-inch bolts at minimum value will develop 590 kip-inches, or 113 per cent M_p. Elongation readings just prior to failure, when converted to loads using the bolt test curve, Fig. 35b, indicate a moment of 582 kip-inches or 112 per cent M_p. It should be noted that the grip of the bolts in the beam test was increased to 2 1/4-inches by use of washers because of insufficient threaded length.

Rotation.--The rotation obtained at yield of the connection plate was 18 times the yield rotation of the beam. The rotation curve, Fig. 22a, tends to be misleading for this test as no rotation readings were taken after reaching a moment of 100 per cent M_p. At this load the connection plate yielded and the beam was considered to have failed. Actually, however, as described below, the beam accepted a much higher load with corresponding greatly increased rotation. Although no rotation data were taken, an appreciation of the final rotation obtained can be gained from Fig. 23. The rotation curve was linear to 60 per cent M_p.

Deflection.--The deflection at yield of the connection was 0.44 inch or 3.3 times the theoretical beam deflection, which is 0.134 inch at yield of the outermost fibers. The test deflection at a comparable load was 0.23

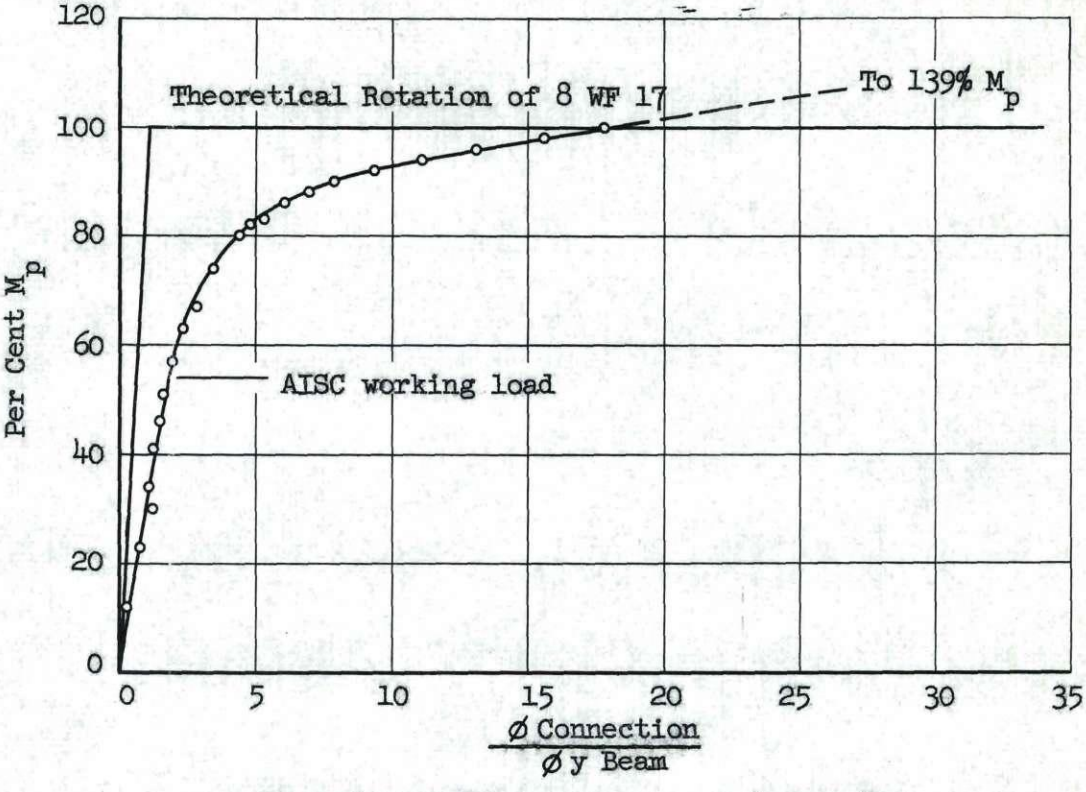

Fig. 22a Moment-Rotation Test 8,17-5-4,7

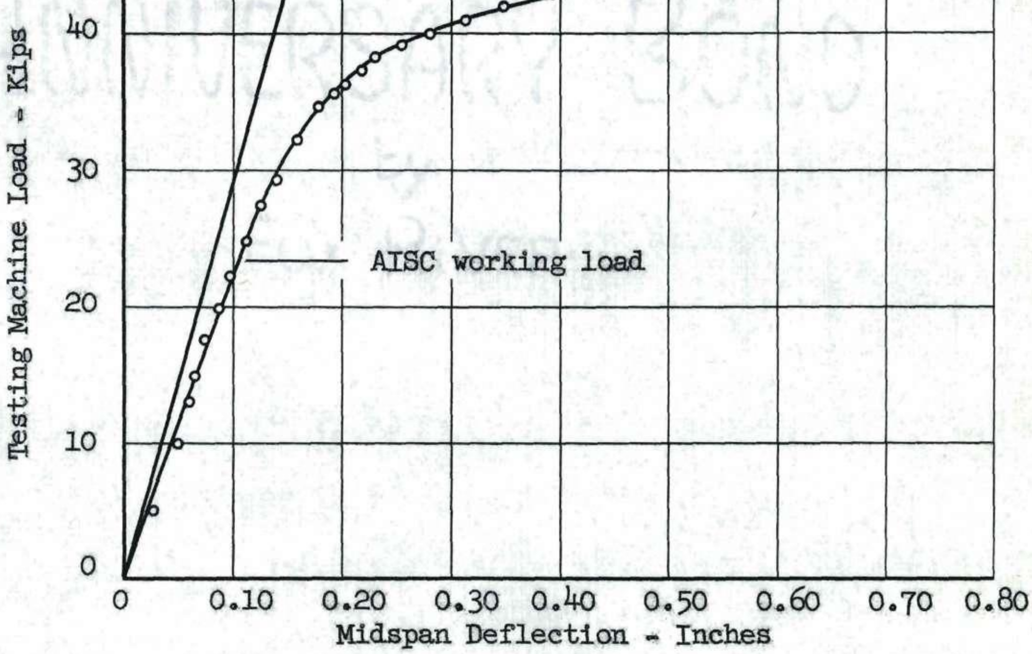

Fig. 22b Load-Deflection Test 8,17-5-4,7

inch or 1.7 times greater. At a stress of 20 kips per square inch in the outermost fiber of the beam, the theoretical beam deflection is 0.081 inch. The test deflection under such a load was 0.11 inch or 1.35 times greater. The results noted for the rotation curve are also applicable to the deflection curve, Fig. 22b.

Loss of contact.--From Equation (9a) the loss of contact of the connection plates would occur at a load of 43 kips. Actual observed loss of contact occurred at 37 kips.

Failure mode.--The specimen accepted a load of 43.5 kips, 100 per cent M_p, at a normal rate of loading. The failure at this point was yielding in the plate on a horizontal line through bolt hole Nos. 2 and 4. At this point the rate of loading was increased to 0.5 inch per minute free head travel and the specimen loaded to 60.4 kips, or 139 per cent M_p, at which time bolt No. 2 failed by thread stripping in the nut. The load dropped and held 44 kips until finally after considerable deformation, bolt No. 4 failed.

Conclusions.--The most obvious results of this test are the further verifications of the correctness of theoretical Equations (11) and (12). The high ultimate load before bolt failure is not considered important as far as the bolts are concerned, as it was due to the abnormal strength of the bolts. However, it is believed of importance that the connection was able to withstand the extreme deformation required to fail these bolts. This deformation occurred in the connection plate, the beam web, flanges, and welds, and was entirely ductile in nature. Figures 23 and 24 show the specimen after removal from the testing machine; the extent of deformation and condition of the connection plate is readily

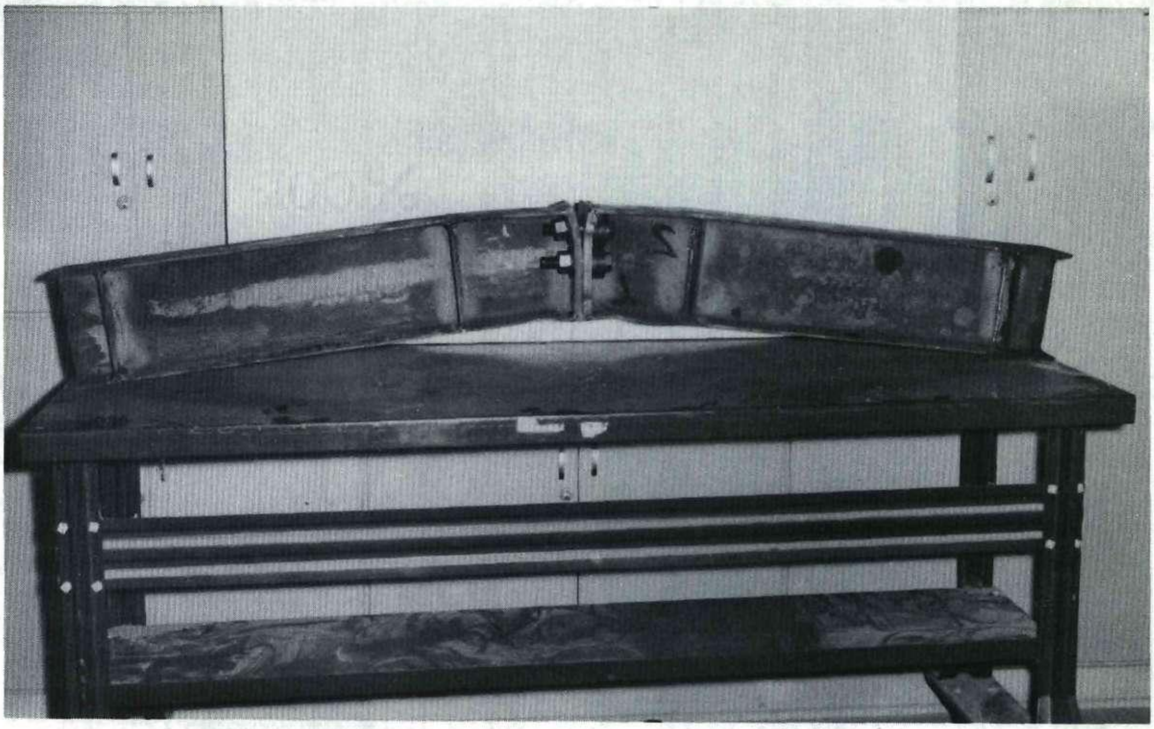

Fig. 23 Test 8,17-5-4,7 After Removal of Load Corresponding to 139 Per Cent M_p

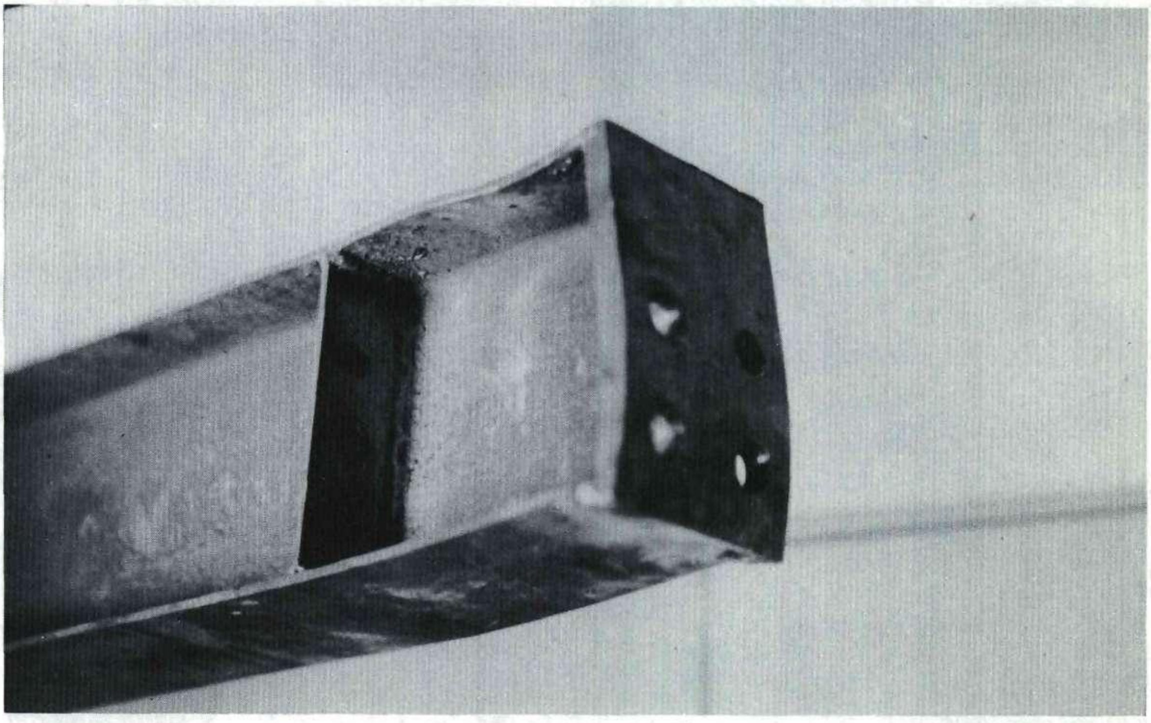

Fig. 24 Test 8,17-5-4,7 After Disassembly

apparent. Of particular note is the failure mode of the plate, with deformation of plastic hinges through bolt hole Nos. 2 and 4 and in the beam flanges at the ends of the connection plate.

The deflection and rotation characteristics of this test must be interpreted with due allowance for the undersize connection plate. Although developing adequate strength, as was expected due to the higher than minimum yield apparent from test 8,17-5-6,5, the reduced thickness will increase the rotation and deflection in proportion to the plate moment of inertia. The majority of the increased deflection of this test specimen as compared to the theoretical beam deflection may be attributed to the connection plate, as the bolt elongations were negligible at the plate yield load. Nevertheless, the magnitude of deflection at working loads was only slightly in excess of the theoretical beam deflection.

Specimens were cut from the beam web and flanges and tested. Results of these tests are included in the Appendix; they show an average lower yield of the beam flanges of 36.8 kips per square inch. The average lower yield of the web was 40.2 kips per square inch. Assuming the entire beam to have a yield of 36.8 kips per square inch would result in a plastic moment of 582 kip-inches, and this would require a testing machine load of 46.8 kips. This information shows that the beam was virtually in a fully plastified condition at yielding of the connection plate. The 12 per cent difference between the moment obtained from the testing machine load and the moment obtained from Equation (12) using values of bolt forces from elongations and Fig. 35b, is explained as in test 8,17-5-6,5. That is; this error is believed due to a drop off in loads of bolt Nos. 1 and 3. The elongations of these bolts were not monitored near ultimate beam load and values used in Equation (12) were the last values appearing in Fig. 25. The error in the case of this test is in the opposite sense from the error of test 8,17-5-6,5. If, however, the reduced value for the upper bolt tensions is substituted in Equation (12), it is found that opposite results will be obtained for the two cases. In test 8,17-5-6,5 an increase in moment will result, while in test 8,17-5-4,7 a decrease in moment will result. In the case of both tests, the prediction from Equation (12) will therefore tend to approach the actual moment obtained with a reduction in upper bolt tension, even though the errors are of opposite sign. This reduction in bolt tension is further confirmed by the error between the predicted and observed loss of contact load. The initial bolt tensions were used to compute the predicted load; a decrease in bolt tensions would account for the earlier occurrence of loss of contact.

Test 8,17-5-4,7 was the first to accept sufficient load to cause full plastic moment in the beam, and for this test the connection plate yielded at 100 per cent M_p based on minimum theoretical yield of 33 kips per square inch. This was actually 97 per cent M_p based on the actual yield strength obtained from specimens cut from the beam. Full correlation is not possible between the actual yield of the plate and test results because: 1) the plate was 5/8-inch thick while the theoretical thickness based on plastic moment in the beam at a yield of 33 kips per square inch and a yield value for the plate of 33 kips per square inch would be a 3/4-inch plate, and 2) unfortunately, no specimens were obtained from the connection plate and its true yield is unknown. In spite of this, it can be seen that with a yield value of the plate slightly in

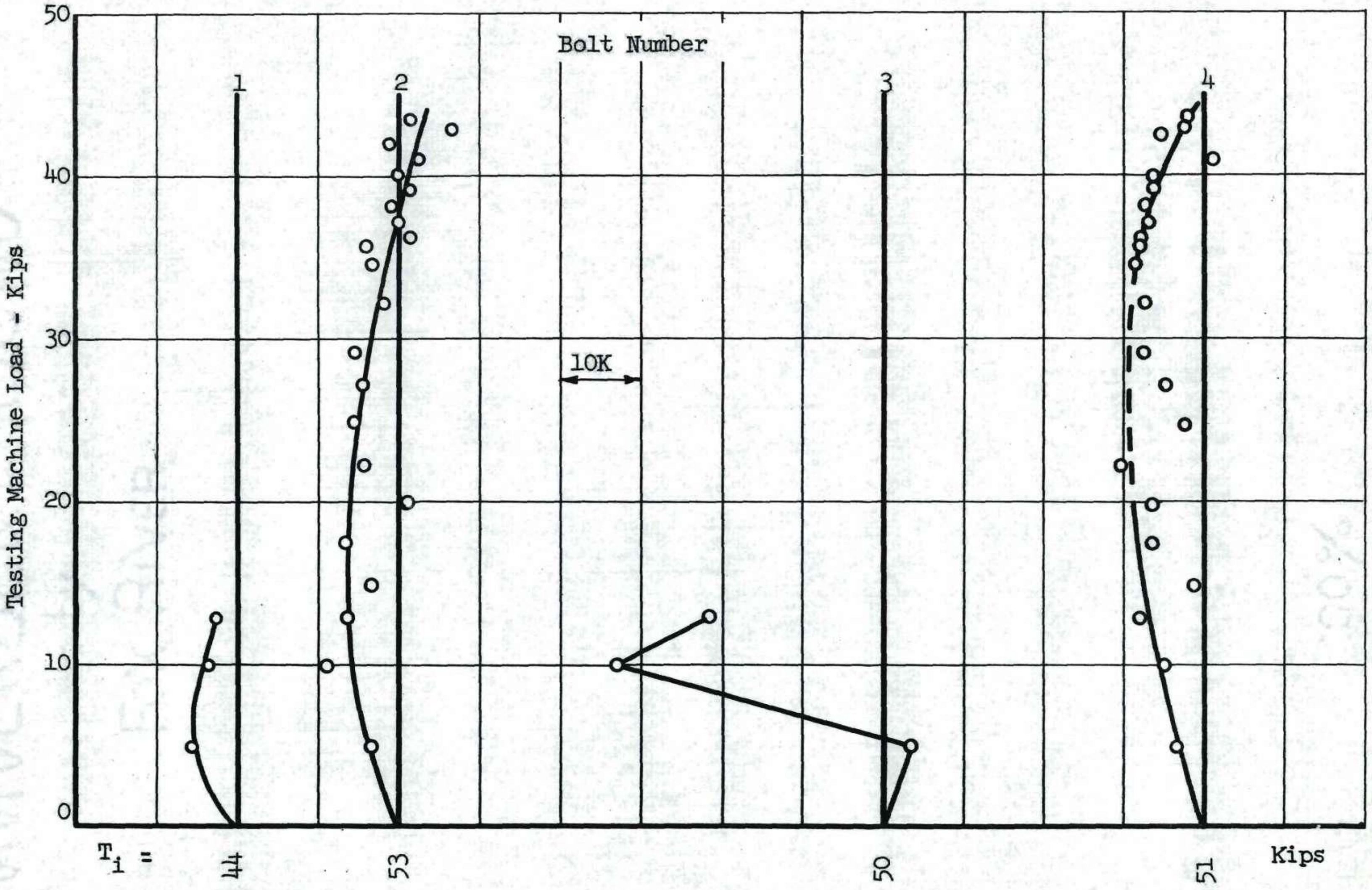

Fig. 25 Bolt Forces - Test 8,17-5-4,7

excess of 33 kips per square inch, the test results would check very closely with the predicted results. The plate carried load considerably past its yield load as in tests 8,20-3-6,5 and 8,17-5-6,5.

Series C, Test 18,50-8-10,7

On study of the overall action of the previous tests and the derived theoretical Equations (11) and (12), it would appear that the most critical connection would occur in the case of a rather deep, narrow beam with a large number of bolts. One question that immediately comes to mind for such a case is the failure mode of the connection plate. In the derivation of Equation (11), the plastic hinge in the connection plate is assumed to occur on a horizontal line through the lower bolt holes. This was found to be true for previous tests, but these tests were for sections having widths not greatly less than their depth, and involving, at the most, three bolts in a vertical line. Another question that may be raised concerns the action of the bolts when used in a deep beam. Will their failure mode be different?

The final test was proposed to study a case of a relatively deep, narrow beam with a larger number of bolts on a vertical line. While the beam used is not an extreme case, the ratio of depth to width for the 18 WF 50 beam is 2.4; and previous tests had ratios of the order of 1.6. The largest depth-width ratio for rolled WF sections is of the order of 3.

It was also desired to conduct this test with a connection plate thickness and a number of bolts that would exactly satisfy the theoretical Equations (11) and (12). This was done for the connection plate thickness as the theoretical thickness is 0.97 inch and a one-inch plate was used. The selection of the bolts was somewhat arbitrary, and is better explained by reference to Table 3.

Table 3. Selection of Bolts for Test 18,50-8-10,7

Bolt diameter*	No.	q	p	T_i in kips	T_u in kips	Per cent M_p of 18 WF 50
7/8	10	2	2 3/8	31.6	53.2	72
7/8	12	2	2 3/8	31.6	53.2	70
1	6	2	2 5/8	42.5	69.7	77
1	8	2	2 5/8	42.5	69.7	89
1	10	2	2 5/8	42.5	69.7	89
1	12	2	2 5/8	42.5	69.7	83
7/8	10	2	2 3/8	42.5	69.7	95

*All figures in this table not otherwise noted are in inches.

Ten one-inch bolts are seen to develop 89 per cent M_p if minimum recommended values are used for initial tension and ultimate strength of the bolts. In the case of ten 7/8-inch bolts, only 72 per cent M_p can be developed using the corresponding minimum bolt values. In view of the abnormal strength of the 7/8-inch bolts, actually greater than the minimum requirement for one-inch bolts, it was decided to use ten 7/8-inch bolts torqued to an initial tension equivalent to the minimum requirements for the one-inch bolts. Such an arrangement will actually develop more than the equivalent number of one-inch bolts, due to the reduced spacing of the 7/8-inch bolts; this value is 95 per cent M_p. It was expected that the ten 7/8-inch bolts would actually develop more than 100 per cent M_p as the ultimate strength of these bolts was higher than the minimum required ultimate for one-inch bolts. As in some of the previous tests difficulty was encountered in torquing the bolts to the exact required elongation. Figure 26 shows the actual variation. The average initial tension was 47 kips rather than the desired 42.5 kips.

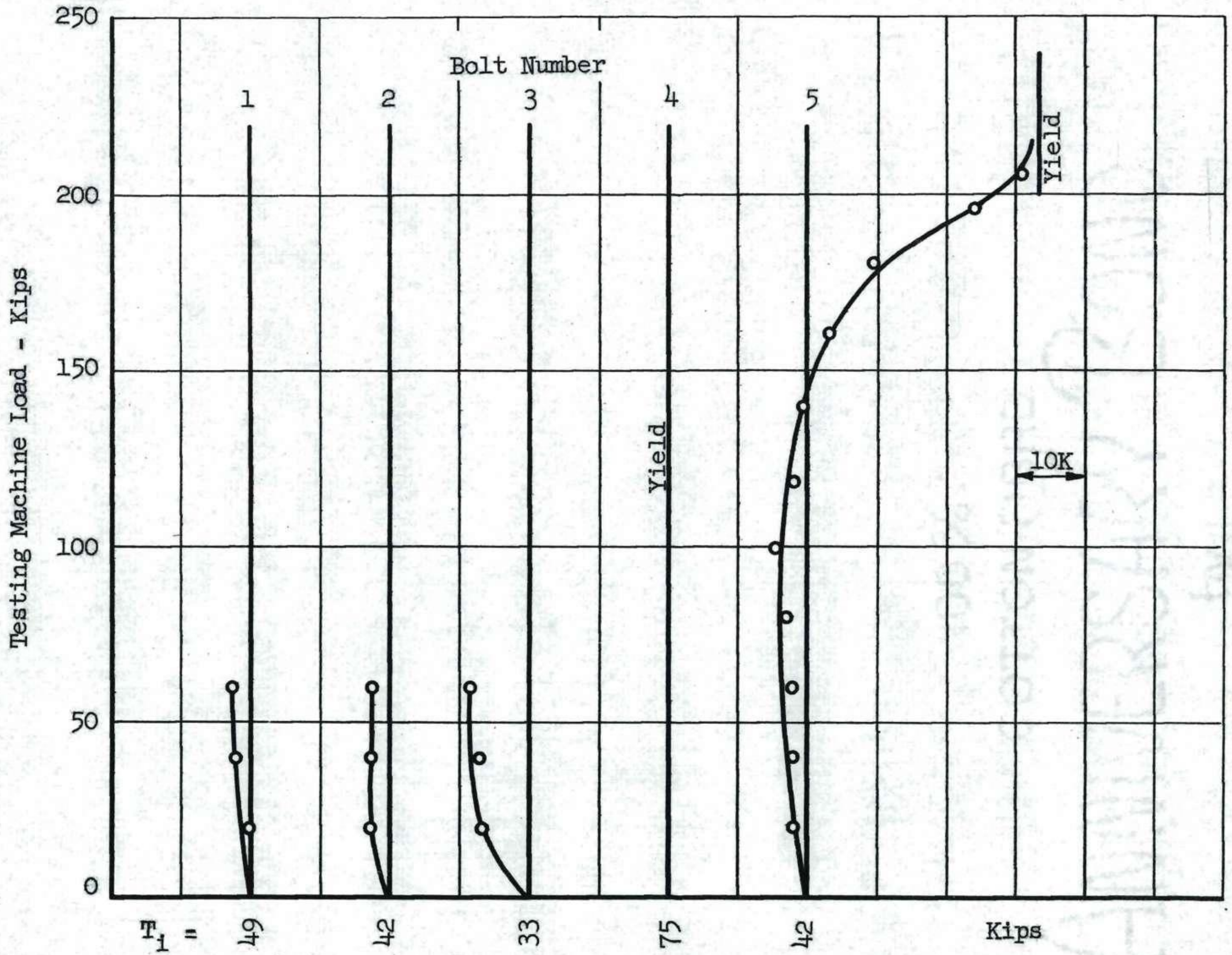

Fig. 26a West Bolt Forces - Test 18,50-8-10,7

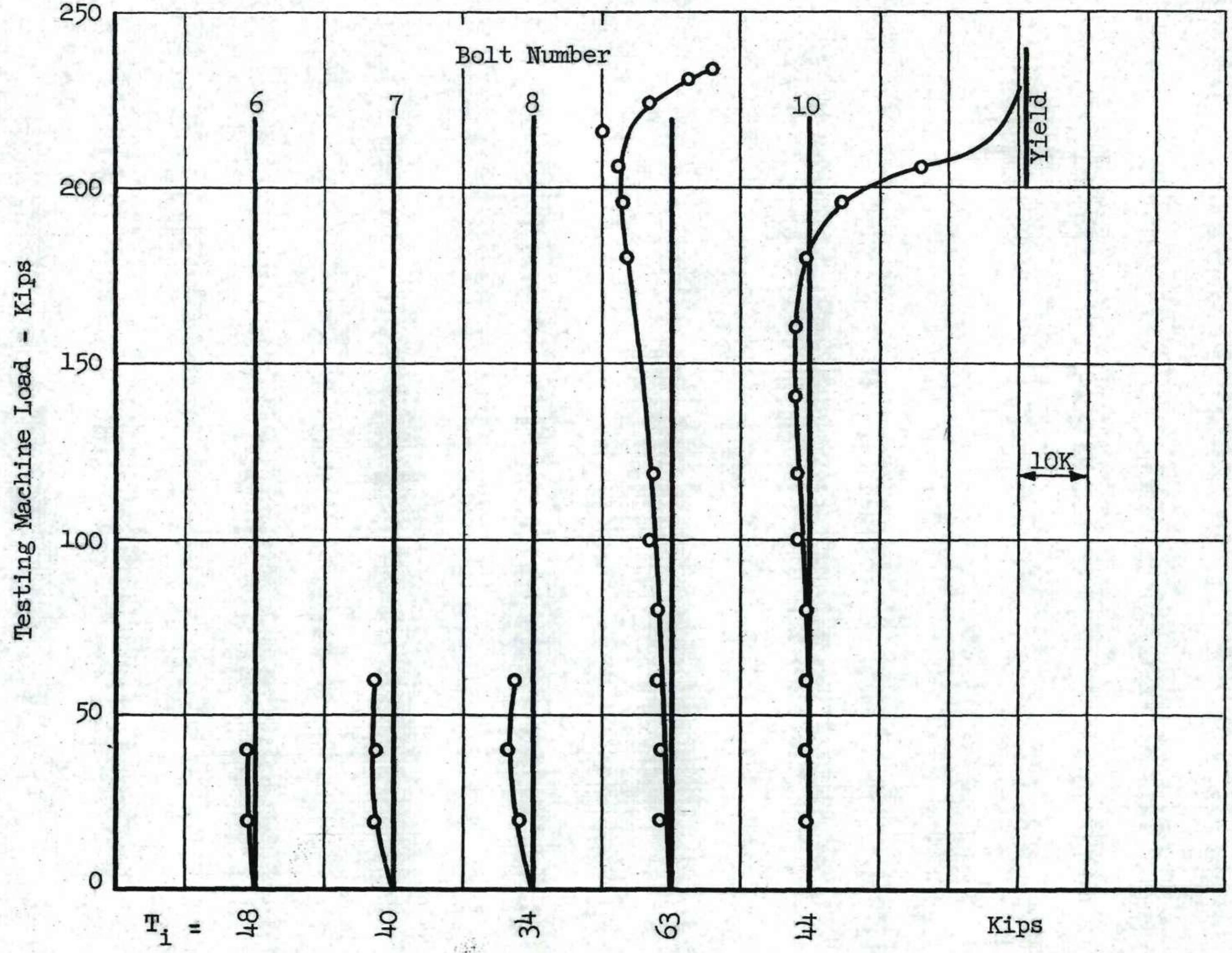

Fig. 26b East Bolt Forces - Test 18,50-8-10,7

Parenthetically, Table 3 illustrates an apparent phenomenon of the direct moment connection that was not discussed in detail with the derivation of Equation (12). As the number of a given size bolt is increased in a connection, a point is reached where the moment capacity seems to decrease. This will occur when $\frac{2}{3}(d + \frac{q}{2})$ becomes less than $q + np$ since $\Delta M = \left[\frac{2}{3}(d + \frac{q}{2}) - (q + np)\right] T_1 R$ for the addition of the nth bolt in a row. If this is true, two solutions are possible: 1) increase the size of the bolts and use fewer bolts, and 2) decrease the bolt size and place the greater number of bolts in four, rather than two, rows. The latter may involve clipped washers (7) in order to fit the connection plate. Actually, this apparent decrease is believed to be due to simplifying assumptions in the derivation of Equation (12) and not a true phenomenon of the connection.[5]

Moment developed.--The test ultimate moment developed was 105 per cent M_p. Bolt loads obtained from elongation readings just prior to failure and the bolt stress-strain curve, Fig. 35b, when substituted in Equation (12), gives a calculated moment of 104 per cent M_p.

Rotation.--The ultimate rotation was 29.4 times the yield rotation of the beam. The rotation at American Institute of Steel Construction working load of 20 kips per square inch was two times the theoretical rotation of the beam and the rotation was linear to approximately 50 per cent M_p which is 3.5 per cent below the American Institute of Steel Construction working load (see Fig. 27a).

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[5]See page 80 for additional discussion.

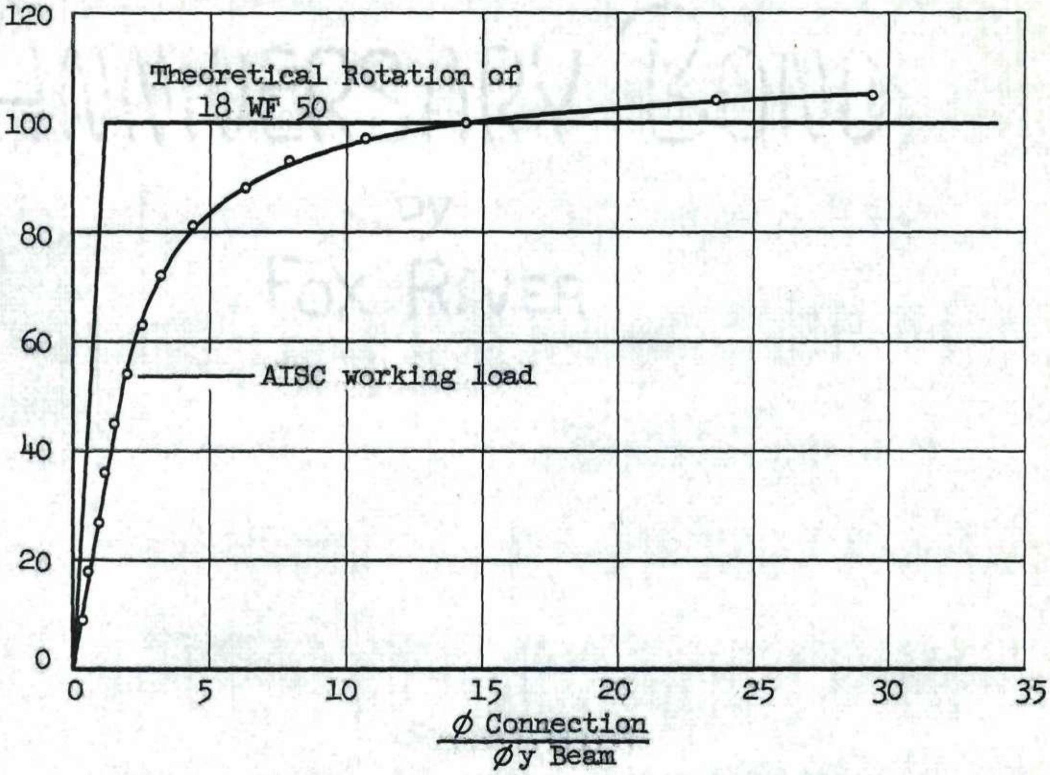

Fig. 27a Moment-Rotation Test 18,50-8-10,7

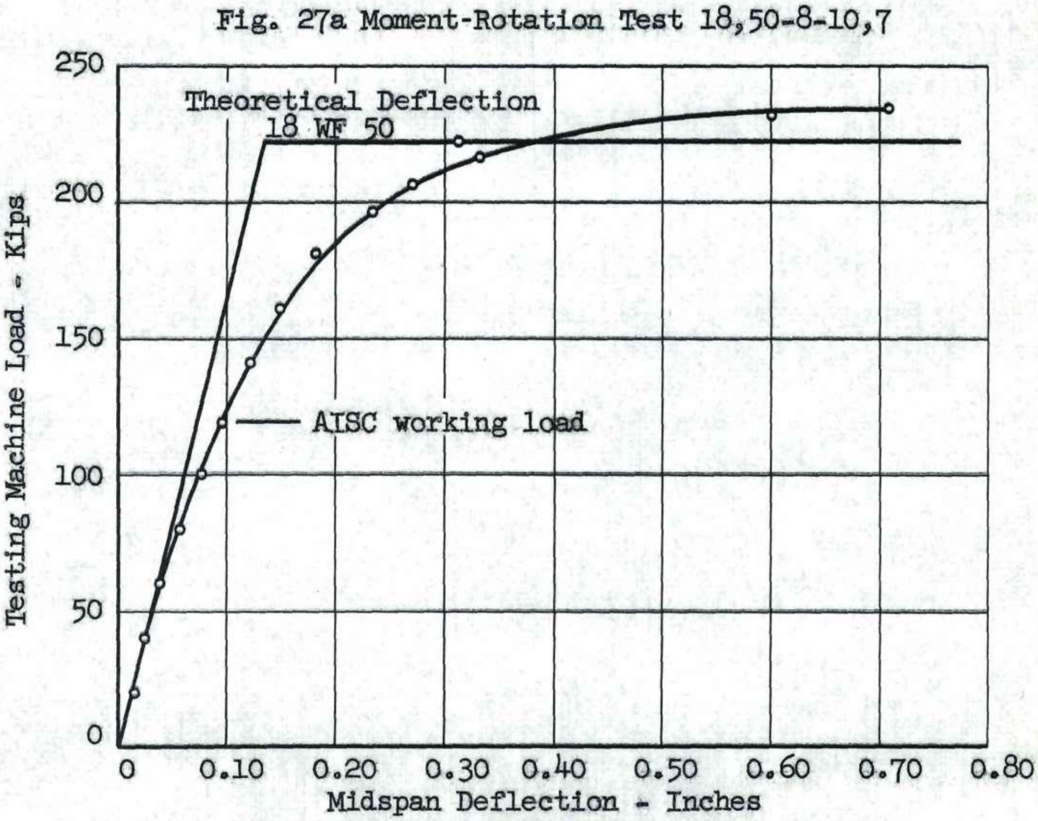

Fig. 27b Load-Deflection Test 18,50-8-10,7

Deflection.--The ultimate deflection was six times the deflection of the beam at yield. The deflection of the test specimen at an outermost fiber stress at 20 kips per square inch was 1.34 times the theoretical beam deflection (see Fig. 27b).

Loss of contact.--From Equation (9a) the loss of contact of the connection plates would occur at a load of 170 kips. The observed loss of contact occurred at 160 kips.

Failure mode.--Initial plate separation occurred at a moment of 2380 kip-inches, or 72 per cent M_p. Yielding of beam flanges, observed by flaking whitewash, occurred at a moment of 2720 kip-inches, or 82 per cent M_p. The separation of the connection plates advanced to bolt Nos. 5 and 10 at a moment of 3150 kip-inches, or 95 per cent M_p. Yielding of the connection plate occurred on a horizontal line through bolt hole Nos. 5 and 10 at a moment of 3330 kip-inches or 100 per cent M_p. The rate of loading was increased and a maximum load of 249 kips, or 112 per cent M_p, was reached, which immediately dropped to 234 kips, a moment of 3500 kip-inches, or 105 per cent M_p. At this load the nut stripped on bolt No. 5. The rate of loading was again increased; the specimen accepted 249 kips which caused the stripping of the nut on bolt No. 10, and the load dropped to 220 kips, or 99 per cent M_p. Upon reloading, 231 kips was reached, the load dropped with increasing deformation until the nuts of bolt Nos. 4 and 9 stripped at a load of 206 kips, or 93 per cent M_p. The specimen was unloaded at this point. No data are included, but bolt Nos. 4, 5, 9, and 10 were replaced using double nuts and the specimen reloaded. The specimen again reached a load of 249 kips; deformation increased rapidly in the beam and the final failure was lateral buckling of the beam.

Specimens were cut from the beam and tested. Data are included in the Appendix, and calculations show the actual beam plastic moment to be 3960 kip-inches or 119 per cent of the minimum theoretical plastic moment.

Conclusions.--The connection plate yielded exactly at the predicted load. This actually is somewhat misleading; two standard 505 specimens were prepared from one of the connection plates, and an average lower yield of 36.8 kips per square inch was obtained (see Table 8). Thus an actual error exists; the error is, however, small, as the theoretical plate thickness would be 0.90 inch for the actual yield strength instead of 0.97 inch for the minimum theoretical yield. It has not been previously stated, but the required plate thickness for all tests was computed on the gross plate area with no allowance for bolt holes. This procedure was adopted because it was believed that the restraint and local compressive force supplied by the bolts would account for an increased ability of the plates to develop plastic moment. However, when the actual yield strength of the plate is considered, it appears that computations based on the net plate area will give better results; furthermore, the error will be on the safe side for such a procedure. The increase in plate thickness will also reduce the rotation and deflection.

Good results are obtained when the calculated moment using Equation (12) is compared to the test moment. Equation (12) gives results 1.0 per cent less than the actual moment obtained.

The rotation and deflection curves have more meaning in this test than in the previous tests as both the connection plate and bolts were selected to conform to the theoretical values. No attempt has been made to derive theoretical expressions to predict either rotation or deflection as a function of the connection. Rather, for these preliminary studies, it was decided to merely compare the test results with the theoretical beam rotation and deflection when the connection is designed on a basis of strength. It is, of course, dangerous to extend results of a single test; but if this test is assumed to be representative it would appear that no difficulties will be encountered with rotation or deflection if the connection is designed for strength. The rotation curve may be misleading unless it is remembered that the rotation plotted is actually unit rotation, computed from a short gage length across the connection. Figure 27a indicates a rotation of the connection at working load about twice the rotation of the beam. However, since the connection rotation extends over a short length, the contribution of the connection to deflection is not in the same ratio. Most significant is the large amount of rotation obtained before failure; more than enough to allow full redistribution of moment for limit design solutions.

As mentioned, the increase in deflection is not in the same ratio as the rotation due to the short distance occupied by the connection. At working loads this deflection is not sufficiently greater than the theoretical beam rotation to be troublesome for most applications. The most serious feature of both the rotation and deflection curves is the departure from linearity at values only slightly in excess of present working loads. Further tests will be necessary to draw accurate conclusions, but it is believed that a nominal increase in connection plate thickness will correct this situation; it is possible that the increased thickness that would be obtained based on the net plate area will be sufficient.

Pictures of this test show very clearly how well the connection withstood loads even after yielding of the plate, which was considered failure. Figure 28 shows the test at 120 kips total load. The current American Institute of Steel Construction working load for this test is 119 kips. Figure 29 is clear indication that even after yielding of the connection plate, the connection is by no means in serious trouble. The beam at the time of bolt failure in bolt Nos. 5 and 10 is shown in Fig. 30; and Fig. 31 shows the beam after additional deformation caused the failure of bolt Nos. 4 and 9, thorough yielding of the beam is evident.

Summary and Correlation of Tests Results

It is always difficult to draw definite conclusions from a small number of tests. This is magnified when tests are conducted in a new region with no previously reported results to act as a guide. Because of this, the entire study was necessarily a pilot program and the results must be viewed with this in mind.

Only the final test, 18,50-8-10,7, completely fulfilled the theoretical requirements to develop the plastic moment of the beam. With regard to the connection plate thickness, the other four tests are valuable more qualitatively than quantitatively. This is particularly true of the variation in connection plate thickness as it affects the rotation and deflection. The composite moment-rotation curve, Fig. 32, shows the trend of action of the connection as the theoretical values of connection plate thickness and bolt size are approached. The increased strength and decreasing rate of rotation are apparent. That the final test, designed to fully develop the beam, did so with adequate rotation and only little

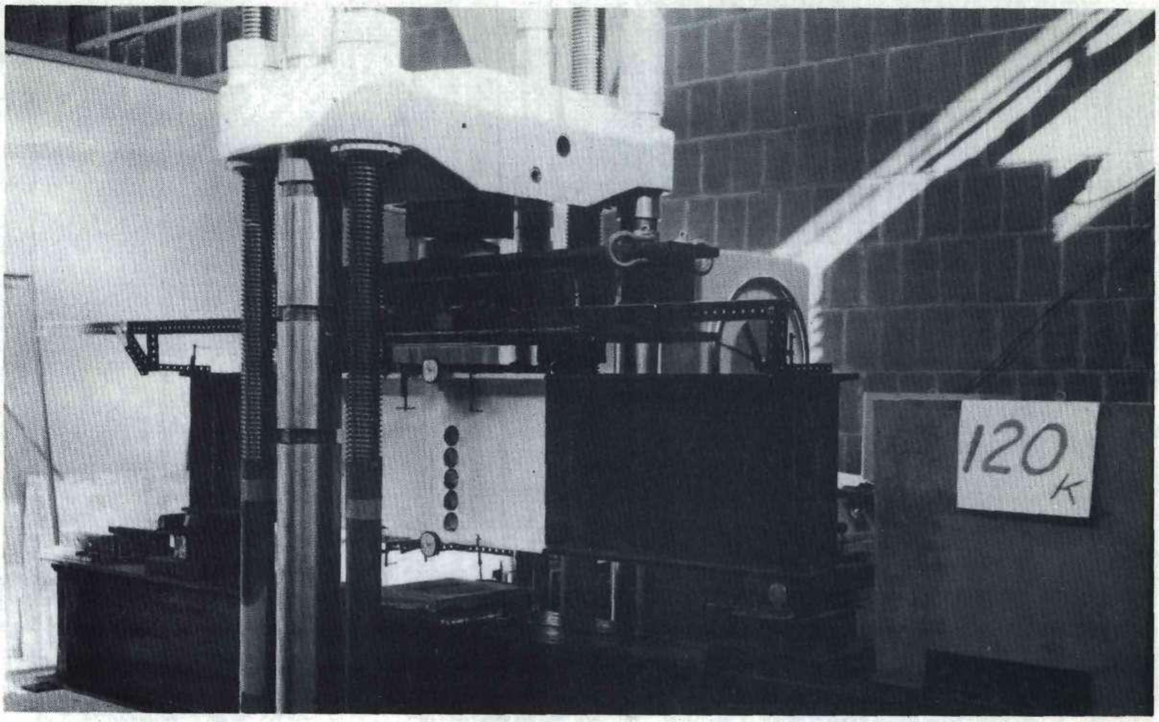

Fig. 28 Test 18,50-8-10,7 at Current AISC Working Load

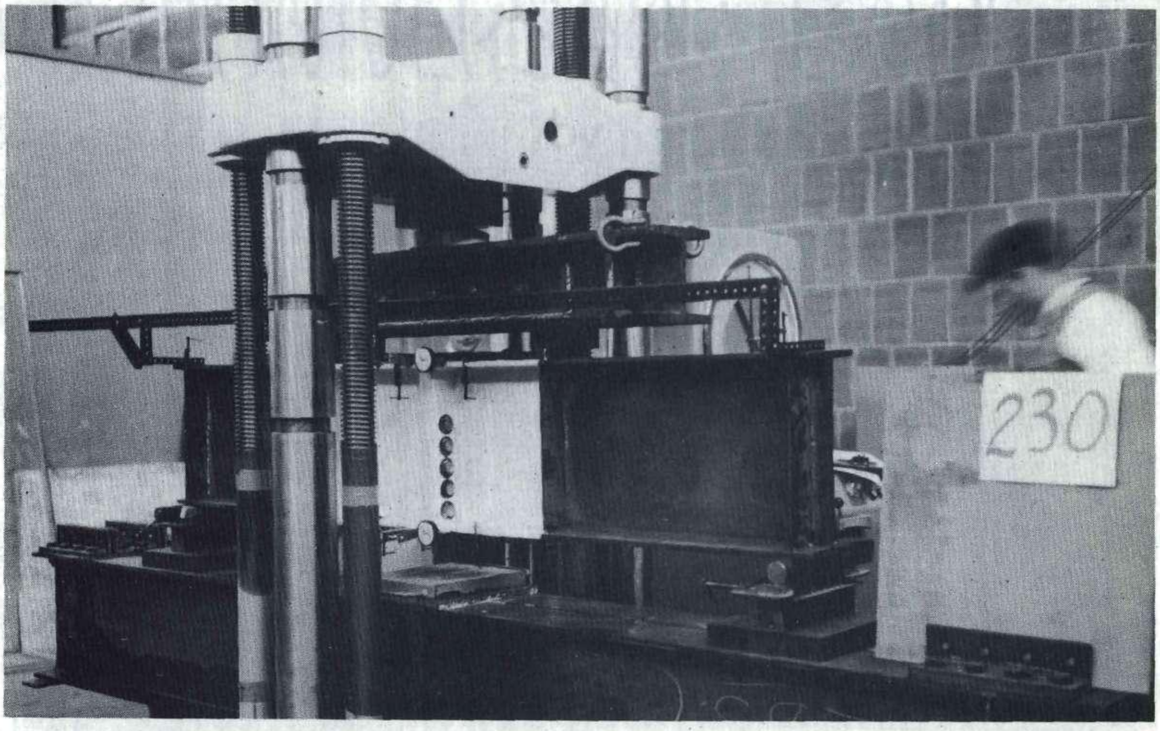

Fig. 29 Test 18,50-8-10,7 Eight Kips Above Yield Load of Connection Plate

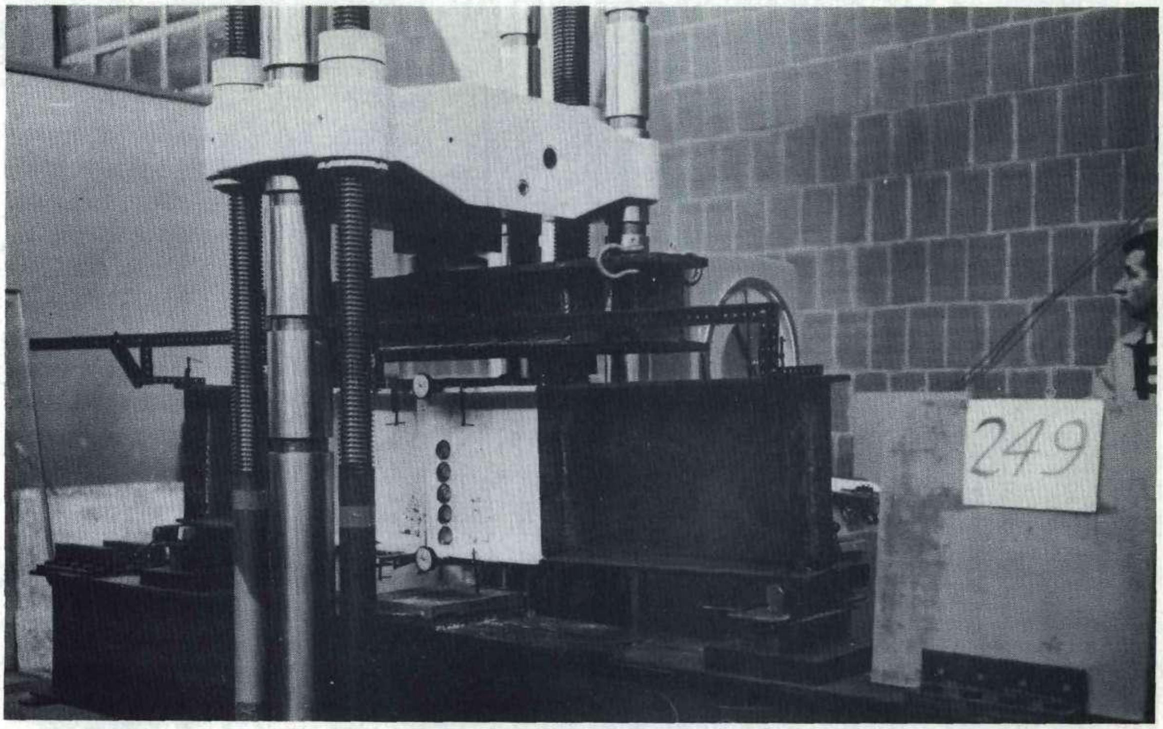

Fig. 30 Test 18,50-8-10,7 at Load Causing Bolt Failure of Bolts 5 and 10

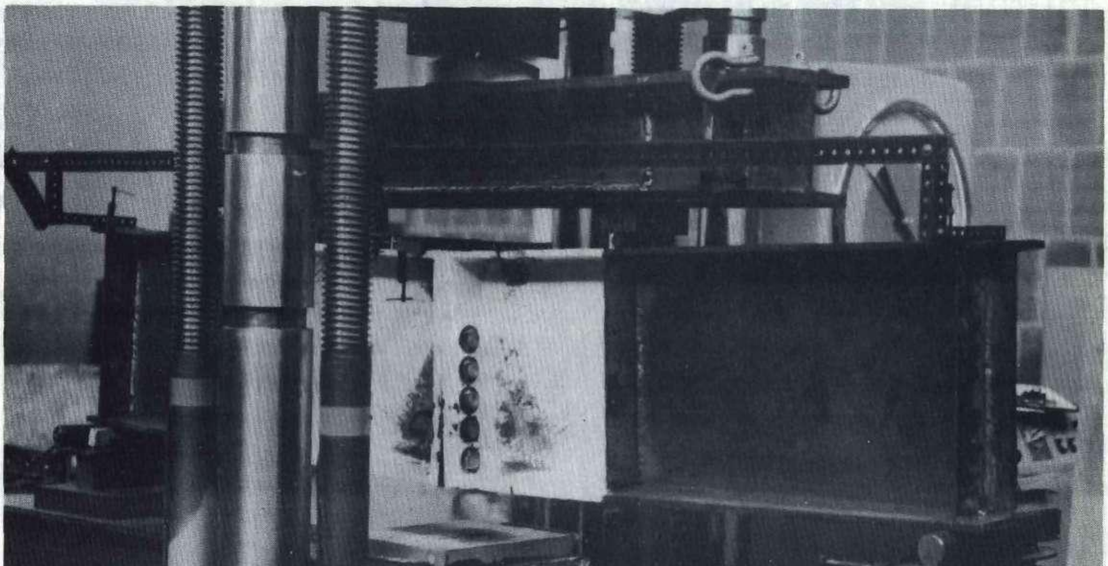

Fig. 31 Condition of Test 18,50-8-10,7 at Failure of Bolts 4 and 9

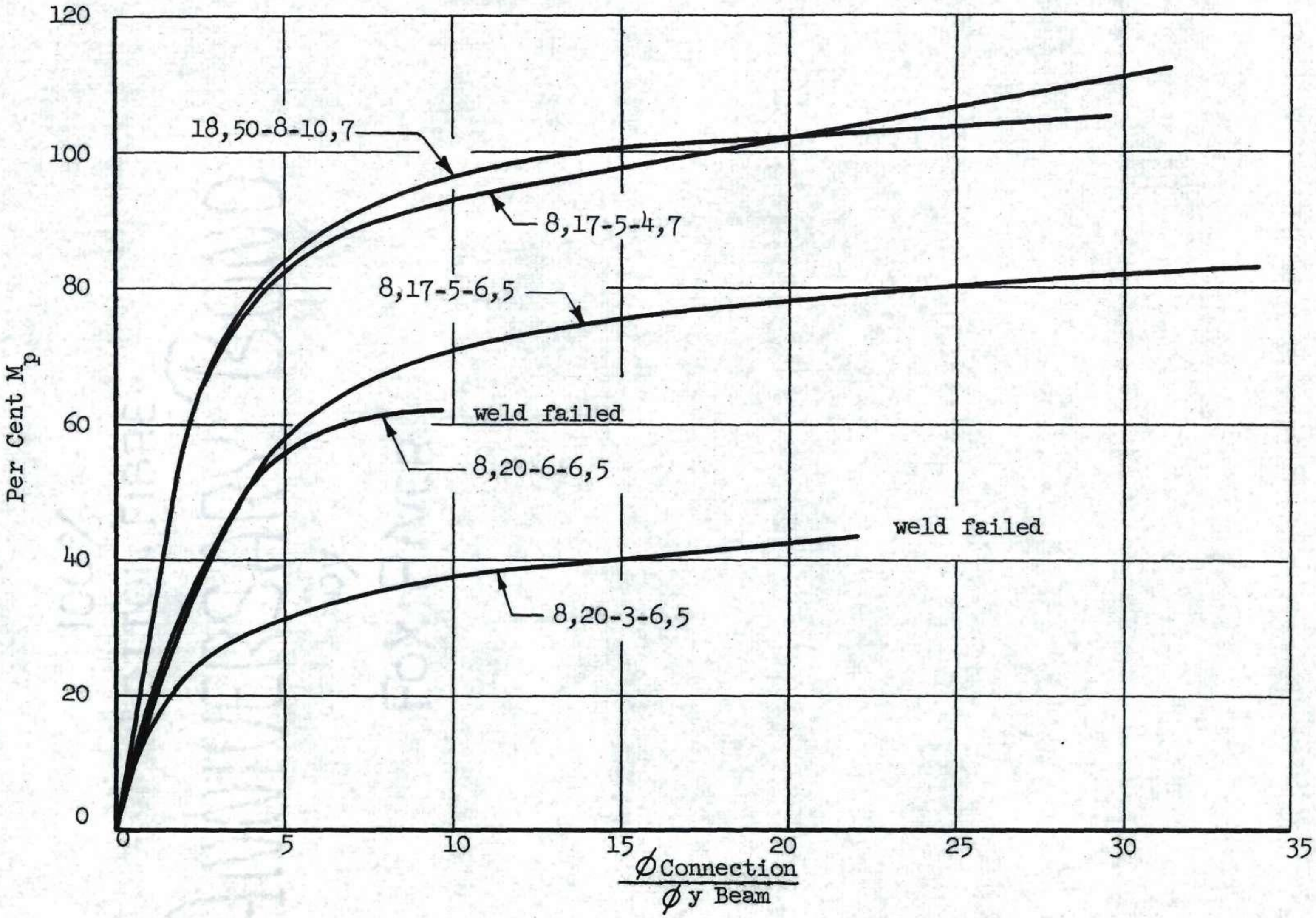

Fig. 32 Composite of Moment-Rotation Curve

increase in deflection is not, as one test, conclusive by any means. But when viewed with the previous four tests which had progressively stronger connections with due allowance for the weld failure of two tests, more reliability may be assigned to the final test. This is particularly true when the close agreement of the moment rotation curves of test 8,17-5-4,7 and test 18,50-8-10,7 is noticed. Test 8,17-5-4,7 met the requirements of the theoretical equations except for the decreased connection plate thickness which was likely almost fully compensated for by a higher than minimum yield strength. The rotation and deflection data is summarized in Table 4.

Table 4. Summary of Rotation and Deflection Data

Test	Connection ultimate rotation / Beam yield rotation	Connection rotation / Beam rotation *	Test deflection / Theoretical beam deflection *
8,20-3-6,5	22	--	--
8,17-5-6,5	34	7.5	1.65
8,17-5-4,7	--	3.0	1.34
18,50-8-10,7	29	2.0	1.34

*At AISC working load of 20 ksi in beam.

No attempt was made to correlate quantitatively the initial yielding of the connection plates for tests in which the beam was not at least near the plastic moment. In view of the method used for derivation of Equation (11), in which the beam was assumed to have developed full M_p, it appears illogical to correlate data from tests not in such a condition. Test 8,17-5-4,7 and test 18,50-8-10,7 did reach a plastified condition as was discussed in the conclusions of these individual tests, the action of the connection plate indicated Equation (11) to be essentially correct, although the thickness would appear to be better selected when the net area of the plate is used rather than the gross area.

Agreement of the moment developed as found from measured elongation of the bolts and Equation (12) with the actual moment computed from the testing machine load is somewhat erratic if viewed coldly. A summary of this data is presented in Table 5. Because of the early weld failure, test 8,20-6-6,5 is considered meaningless, and is not included in this Table. This is particularly true with regard to correlation of bolt data as the connection plate behaved in an entirely different manner from that assumed in the derivation of Equation (12).

Table 5. Comparison of Test Moment and Moment Computed from Equation (12)

Test	Moment at ultimate from testing machine load in per cent M_p	Moment computed from bolt elongations, Fig. 35, and Eq. (12) in per cent M_p	Error in per cent
8,20-3-6,5	43	45	+2
8,17-5-6,5	82	63	-19
8,17-5-4,7	100	112	+12
18,50-8-10,7	105	104	-1

Equation (12) involves two known approximations. The first of these was the assumption that all bolts except those immediately adjacent to the tension side of the beam remain at the initial tension value as moment is applied. It was expected that forces in other bolts near the tension side would increase near the ultimate moment. This variation was omitted from Equation (12) because: 1) the increase was believed to be small, 2) the equation was greatly simplified as the amount of increase in bolts above the lower set depends on many variables, and 3) the error produced would be on the safe side.

The second approximation was the assumption that the contact pressure has a straight line variation; this is improbable. Near ultimate it seems reasonable that the contact pressure would tend to shift toward the compression side of the beam. If such a situation occurs, the effect would be an increase in the lever arm between the C and ΣT forces and a corresponding greater moment capacity. This effect was, however, omitted from Equation (12) for essentially the same reasons as given above.

This shift of the contact pressure or variation from a straight line also would explain the apparent decrease in moment capacity which occurs when the term $\frac{2}{3}(d + \frac{q}{2})$ becomes less than $q + np$. Most likely a shift in the resultant of the contact pressure will occur as the ultimate load is approached, which increases the constant $\frac{2}{3}$ in the term $\frac{2}{3}(d + \frac{q}{2})$, and the moment capacity will not actually be decreased.

It was not expected that the bolts on the compression side would decrease in tension sufficiently to affect the results. In fact, a complete set of bolt readings was taken for the first test, test 8,20-3-6,5, and the elongation changes in bolt Nos. 1, 2, 4, and 5 were negligible, as shown in Fig. 18. Further, the moment computed from the testing machine load and the moment computed from the bolt elongations and Equation (12) checked within 1 per cent. It was thus believed that the assumptions were justified. This was indeed unfortunate as the upper bolts were not monitored for the remaining tests. The writer, on review of the test data, now believes the differences between test moments and moments computed from Equation (12) are primarily due to an unknown decrease in tension of these upper bolts. This explanation is supported by: 1) the close agreement of test 8,20-3-6,5 in which the upper bolts were monitored, and 2) the fact that although the errors of test 8,17-5-6,5 and test 8,17-5-4,7 are of opposite sense, they would each be compensated by a decrease in upper bolt tensions. Contrary to the above explanation is the small error of test 18,50-8-10,7. Upper bolts were not monitored during this test past 27 per cent M_p, and the lack of error is difficult to accept if the explanation for errors in other tests is true. However, the change in bolt elongation is a function of plate stiffness, beam yielding and initial "fit up" of the connection plate, and it is entirely possible that the majority of the loss had occurred in the bolts of test 18,50-8-10,7 at 27 per cent M_p.

The curves of the various bolts are best viewed from the standpoint of trends since complete data were not taken. A review of Figs. 18, 20, 25, 26a, and 26b, which are plots of the bolt tensions against testing machine load, shows essentially the same bolt action in all tests. First, there is a drop off in bolt tensions in virtually all of the bolts under the initial application of moment to the specimen. This is undoubtedly due to the "seating" of the plates. Second, after this initial drop off, the action of the bolts depends on their location in the connection. As seen in Fig. 25, of test 8,17-5-6,5; Fig. 18 of test 8,20-3-6,5; and Figs. 26a and 26b of test 18,50-8-10,7, the lower bolts lose very little initial tension and soon, in fact, pick up additional tension, finally failing in tests 8,17-5-4,7 and 18,50-8-10,7. The bolts immediately above the lower set lose considerably more tension as the failure load is approached until, as the lower set of bolts yields, they also pick up additional tension. This action is apparent in all of the test previously listed, but is best illustrated by bolt No. 9 of test 18,50-8-10,7, Fig. 26b. This phenomenon would not appear to be satisfactorily explained on the basis of "seating" of the plates alone, but is probably due to a reduction in the plate thickness as the plastic hinge forms through the lower set of bolt holes. This explanation is supported by the action of bolts farther away from the lower set as illustrated by bolt Nos. 1 and 4 of test 8,20-3-6,5, Fig. 18. The loss of tension in these bolts is small and there is a tendency to return to the initial tension as the load is increased.

Undoubtedly, all bolts in a connection would increase in load near ultimate if the lower bolts had sufficient elongation before failure. On the basis of these tests, however, the assumption that the lower bolts yield while the uppermost bolts remain at their initial tension as assumed in the derivation of Equation (12), appears to be a reasonable compromise for connections designed to have simultaneous ultimate plate and bolt loads.

Further studies are desirable to completely define the bolt actions as many variables appear to influence the exact action. For example, the curves of bolt Nos. 2 and 4 of test 8,17-5-4,7, Fig. 25 have considerable scatter. This may be partially due to experimental errors previously discussed; but it is believed that the local yielding of the web and plates could account for the scatter, since the bolts in this case were overdesigned, as torqued, and failure of the plate and plasticity of the beam did not occur simultaneously with bolt failure. Data were not taken, but this test actually developed 139 per cent M_p before bolt failure as previously mentioned. Undoubtedly the bolts followed a trend similar to bolts in the other tests, as failure was approached.

CHAPTER V

CONCLUSIONS

The following conclusions are drawn from this study and are necessarily limited to the conditions of this study:

(1) A direct moment connection can be designed to develop the full plastic moment of a wide flange beam.

(2) Indications are that the theoretical equations developed in this study are essentially correct, and will lead to an adequate design.

(3) When designed by Equation (11) using gross plate width the rotation of the direct moment connection is more than adequate to allow redistribution of moments as used in the plastic analysis of a structure.

(4) The ultimate strength of a direct moment connection is more closely associated with the strength of the bolts than the thickness of the connection plate.

(5) A reasonable reduction in the connection plate thickness from the theoretical thickness will not prevent development of the plastic moment of the beam.

(6) The rotation and deflection characteristics of a direct moment connection are influenced more by the connection plate thickness than the strength of the bolts.

(7) Rotation in the immediate area of the connection is twice the

$\frac{M}{EI}$ value of the beam at American Institute of Steel Construction working loads for a connection plate thickness which is adequate from a strength standpoint; that is, the load causing plasticity of the beam will simultaneously cause yielding of the connection plate. However, as the connection rotation occurred over a short gage length, the increase in deflection is approximately one-third.

(8) Using Equation (11), the connection plate thickness will develop the plastic moment of a beam if the gross connection plate width is used for a connection containing only two rows of bolts. It is believed that the deflection will not be greater than the theoretical deflection of the beam if the connection plate thickness is based on net plate width.

(9) Fabrication of a direct moment connection requires no special techniques. Specifically, the welding of the connection plate to the beam is not critical under static loading if ordinary requirements for structural welds are fulfilled.

(10) Overall results of this study indicate that the direct moment connection offers sufficient promise as a practical connection to warrant a complete investigation.

CHAPTER VI

TENTATIVE RECOMMENDATIONS FOR DESIGN OF A DIRECT MOMENT CONNECTION

Until further study can be made to clarify certain questions raised by this study, the following recommendations for design of a direct moment connection are proposed:

(1) Select bolts from Equation (12), using the largest size feasible, but use no less than two bolts per row.

(2) To compensate for possible loss of initial tension select bolts to develop 1.2 M_p; that is, Equation (12a) becomes 1.2 $M_p = M_b$.

(3) If no reversal of moment is present, locate these bolts as close to the tension side of the beam as possible; minimum spacing of bolts may be based on clearance requirements alone.

(4) Ignore the apparent reduction in moment capacity indicated by Equation (12) by the addition of bolts when $\frac{2}{3}(d + \frac{q}{2})$ becomes less q + np; that is, if such bolts are added, say for torsional stability or to increase shear capacity, they may be considered as non-existent for calculating the moment capacity.

(5) Compute shear capacity at working loads based on a coefficient of friction of 0.3 for unpainted faying surfaces of the connection plate and full initial tension for all bolts.

(6) Where deflection is not critical and only two rows of bolts

are used, select the connection plate thickness from Equation (11) on the basis of gross plate width.

(7) If deflection must be limited to the theoretical deflection, or when four rows of bolts are employed; select the plate thickness from Equation (11) on the basis of net plate width.

CHAPTER VII

RECOMMENDATIONS FOR ADDITIONAL STUDY

(1) Obtain additional information on the variation of bolt tension.

(2) Determine the shape and variation of the contact pressure between connection plates.

(3) Study connections with four rows of bolts.

(4) Develop interaction relations for shear and axial loads.

(5) Determine fatigue characteristics.

(6) Test continuous structures and frames employing direct moment connections.

(7) Develop deflection relations.

(8) Extend study to built up sections.

APPENDIX

APPENDIX A

BOLT TEST

The stress-strain curve for the 7/8-inch bolts was developed by subjecting three bolts from the lot used in the beam test to a tensile test. A special testing yoke shown in Fig. 33 was employed. Elongations were obtained as described on page 41. With a zero gap the grip is 2 1/4 inches for this yoke. The nuts were hand tightened before loading. Bolt Nos. 1 and 2 failed by stripping the threads of the nuts. Double nuts were placed on bolt No. 3 and a tensile failure resulted. This was done as there was evidence that the beam bolts, although failing by stripping of the nuts, experienced greater elongation than the test bolt Nos. 1 and 2. It is believed that the bending present in the beam bolts caused a wedging action, which increases the force required to strip the nuts, thereby increasing the bolt elongation. Figure 34 is a view of the three test bolts.

The 5/8-inch bolts from the same lot used in the beam test of this study were previously tested in a similar manner by others (8).

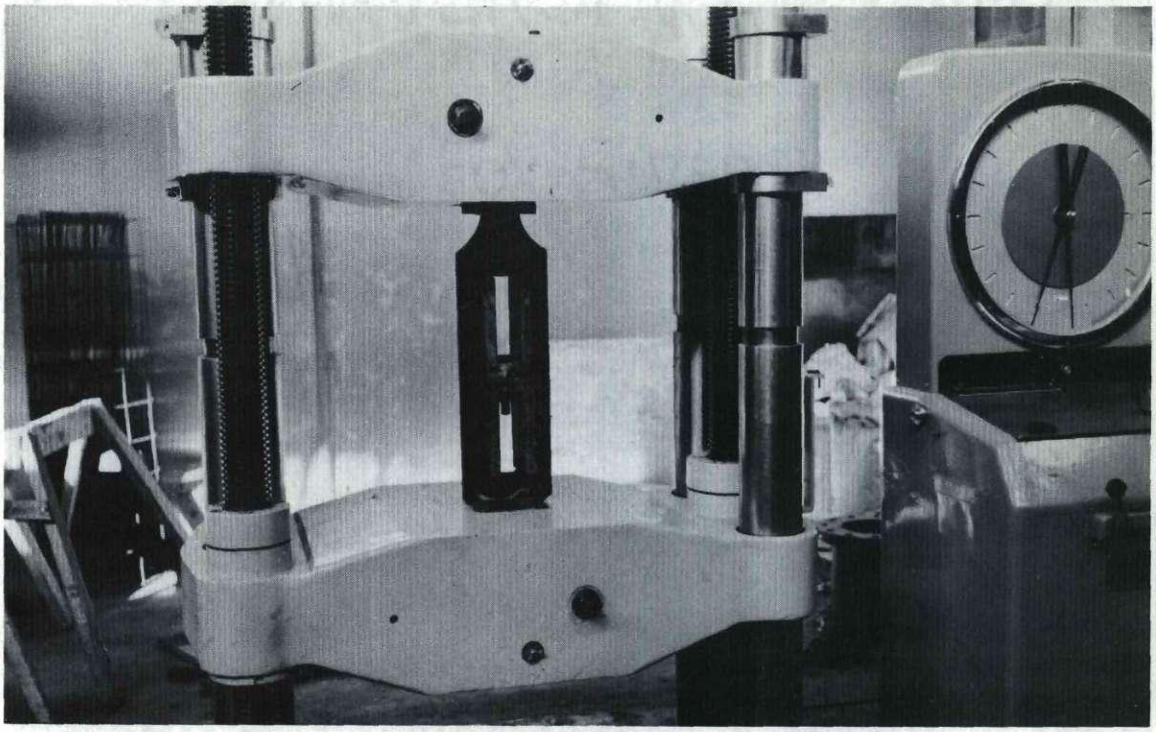

Fig. 33 General View of Bolt Test

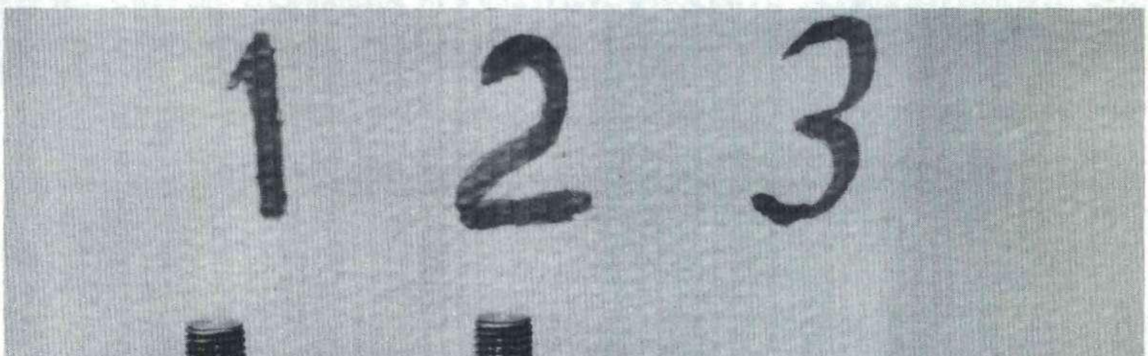

Fig. 34 Seven-Eighths Inch Test Bolts After Loading

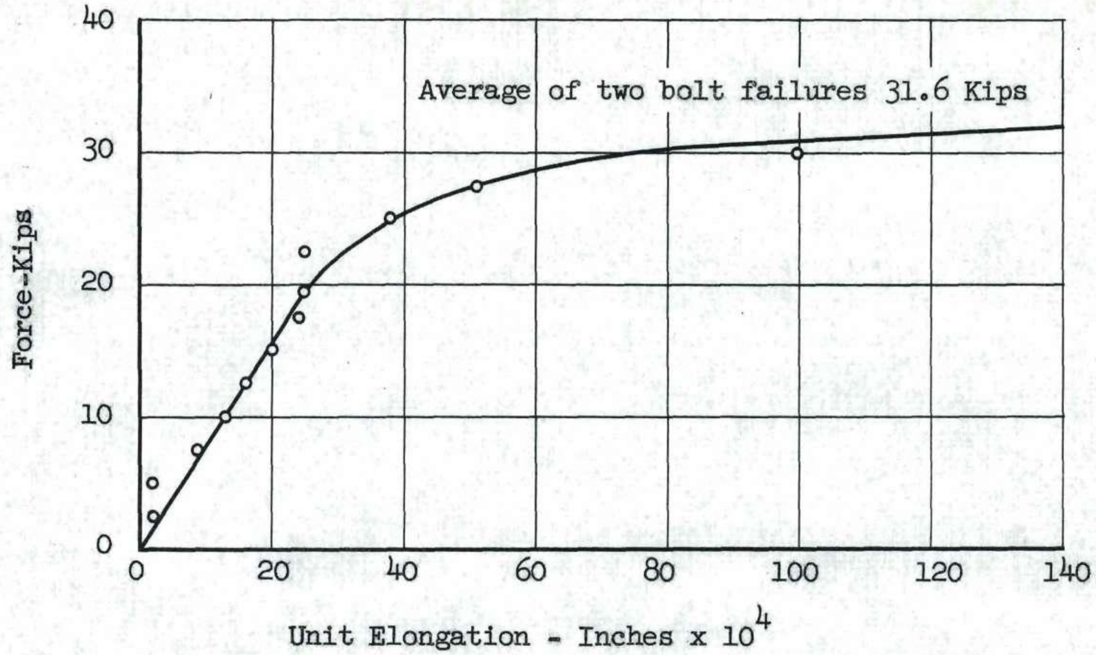

Fig. 35a Stress-Strain Curve for Five-Eighths Inch Bolts

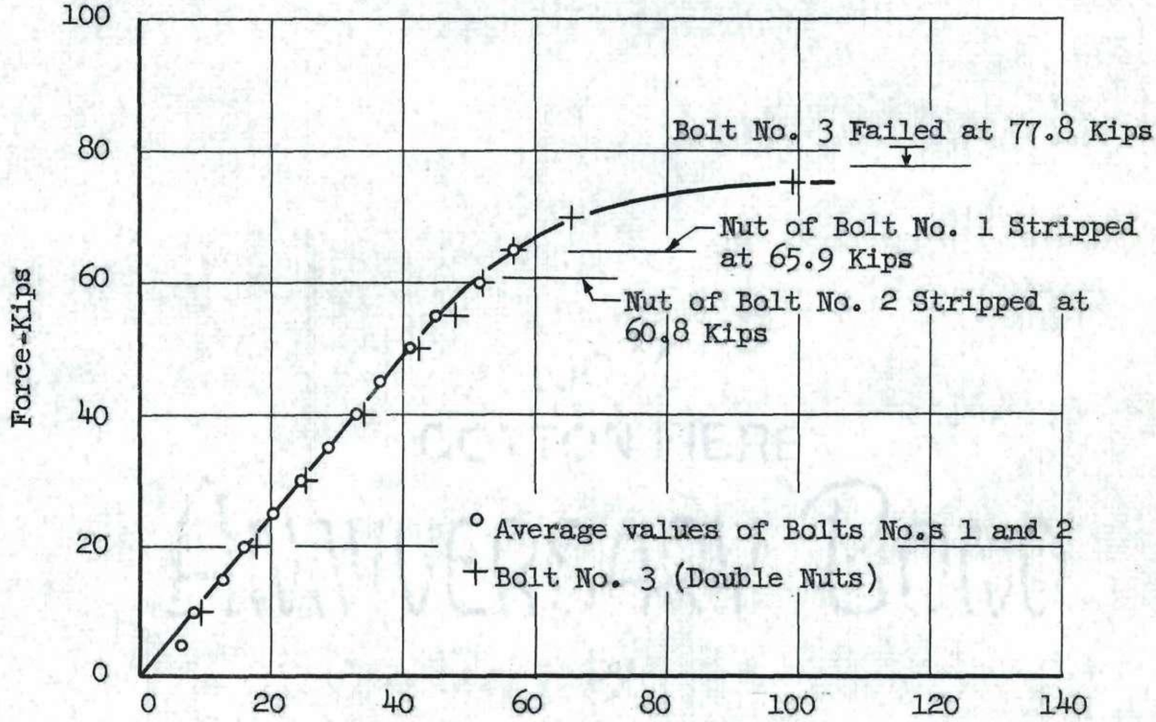

Fig. 35b Stress-Strain Curve for Seven-Eighths Inch Bolts

APPENDIX B

COUPON TEST DATA

Coupons were flame cut from the beams of test 8,17-5-4,7 and test 18,50-8-10,7 and from one of the connection plates of test 18,50-8-10,7. The specimens were machined to the dimensions given in Tables 6, 7, and 8, and tested in tension using an Olsen Super L hydraulic testing machine of 120 kip capacity. The lower yield points were obtained at near zero strain rate by monitoring a micro-mechanical dial gage attached to the machine head. Elongations and area reductions were calculated from measurements made with a machinist's micrometer.

Table 6. Coupon Data From Beam, Test 8,17-5-4,7

Specimen Location	Avg. dimensions width x thickness in inches	Upper yield in kips per in.^{2}	Lower yield in kips per in.^{2}	Ultimate in kips per in.^{2}	Per cent elongation in 2 in. gage length	Reduction in area in per cent
Flange	0.523 x 0.250	39.0	36.8	62.5	30	56
Flange	0.540 x 0.255	38.4	37.0	62.4	30	58
Web	0.521 x 0.247	38.8	37.8	61.5	32	57
Web	0.505 x 0.241	40.5	39.4	62.5	30	57

Table 7. Coupon Data From Beam, Test 18,50-8-10,7

Specimen Location	Avg. dimensions width x thickness in inches	Upper yield in kips per in.^{2}	Lower yield in kips per in.^{2}	Ultimate in kips per in.^{2}	Per cent elongation in 2 in. gage length	Reduction in area in per cent
Flange	2.027 x 0.483	40.4	37.6	63.6	53	50
Flange	2.221 x 0.517	40.2	37.2	63.6	55	50
Web	2.323 x 0.251	48.9	46.2	67.1	44	39
Web	2.203 x 0.354	48.7	45.8	66.2	45	46

Table 8. Coupon Data From Connection Plate, Test 18,50-8-10,7
(Standard 505 Specimens)

Specimen	Upper yield in kips per in.^{2}	Lower yield in kips per in.^{2}	Ultimate in kips per in.^{2}	Per cent elongation in 2 in. gage length	Reduction in area in per cent
1	36.0	35.5	71.7	35	52
2	37.5	38.0	73.0	33	50

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